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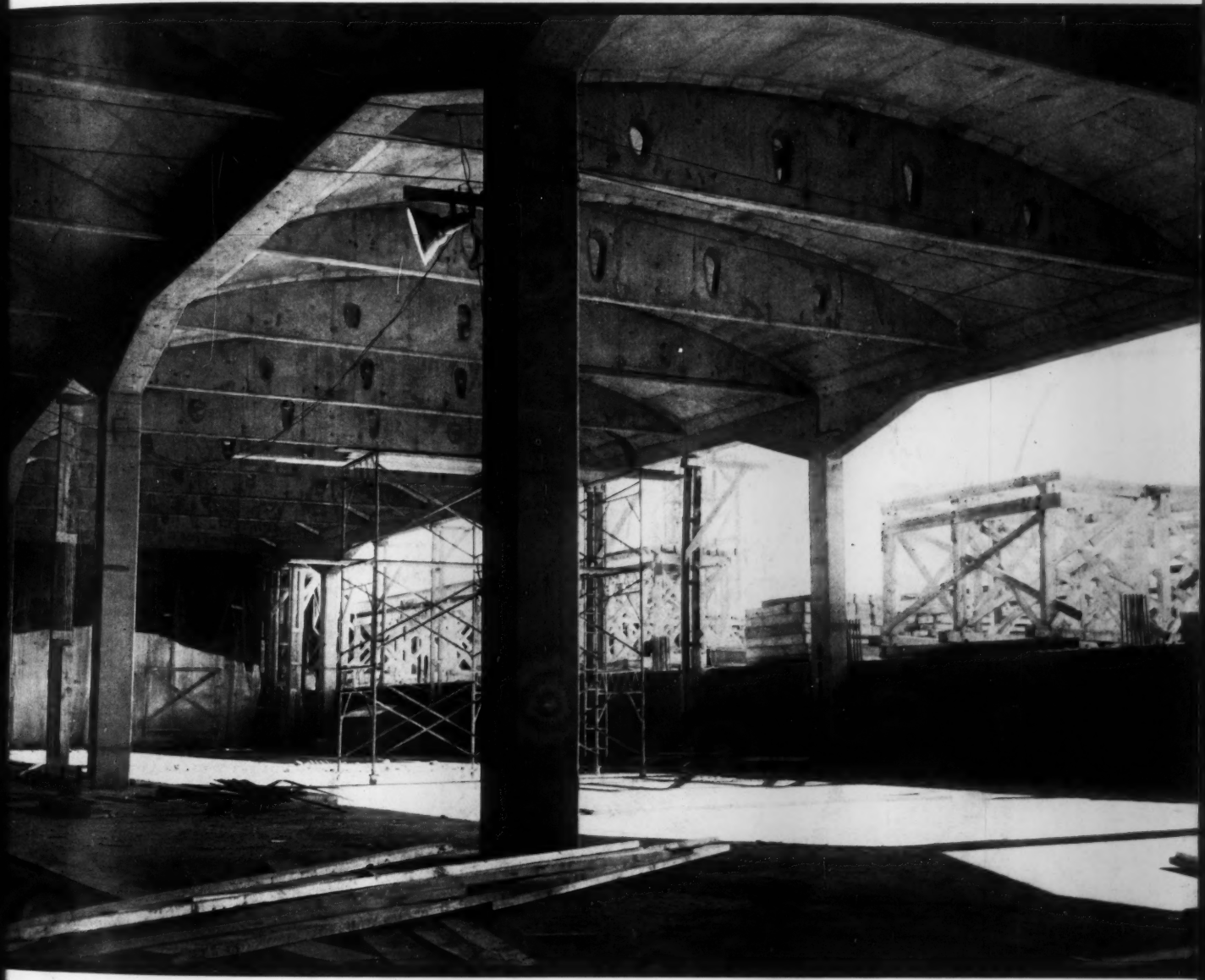
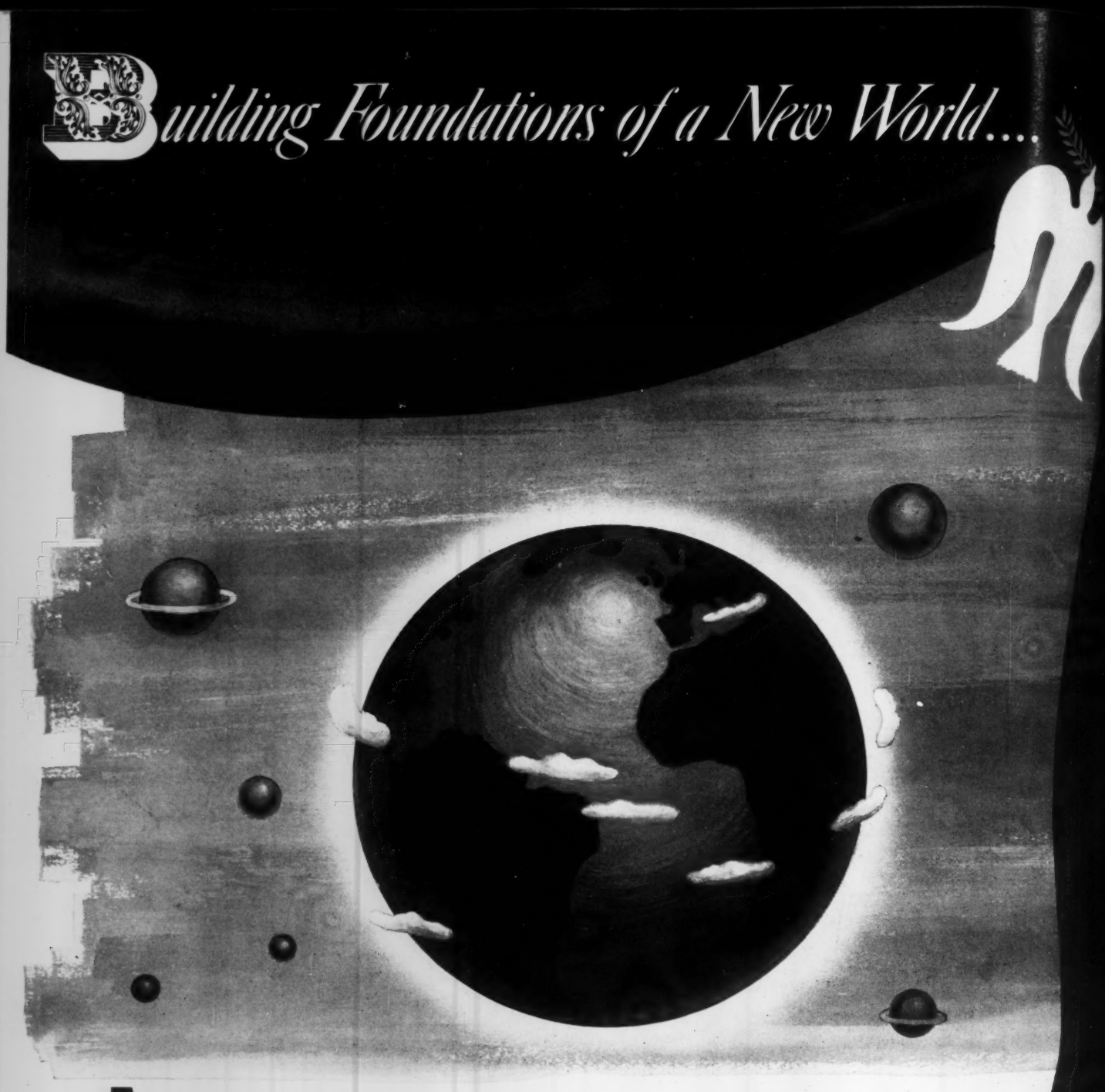


CONCRETE SHELTERS THE WAR BIRDS AT KANSAS CITY AIRCRAFT PLANT
Completed Arch Roof Section (See Article, Page 363).

Volume 13 — *Number 8*

AUGUST 1943

Building Foundations of a New World....



Although engaged in the greatest war in the annals of history, the far-seeing leaders of the United Nations are today formulating plans for establishing a permanent peace on a world-wide basis. We, who write the cartels of the peace to come, must consider the whole world and the rising need of a new, universal philosophy. For, it is fully realized that a steadfast foundation can only be built with the coordination and partnership of all nations. Every country will be called upon to set aright the war-torn world. With malice toward none and with charity for all, the defeated nations must be helped to re-establish themselves so they can eventually reach a better standard of living. With a closer understanding between all peoples of the world, the true dawn of civilization will come.

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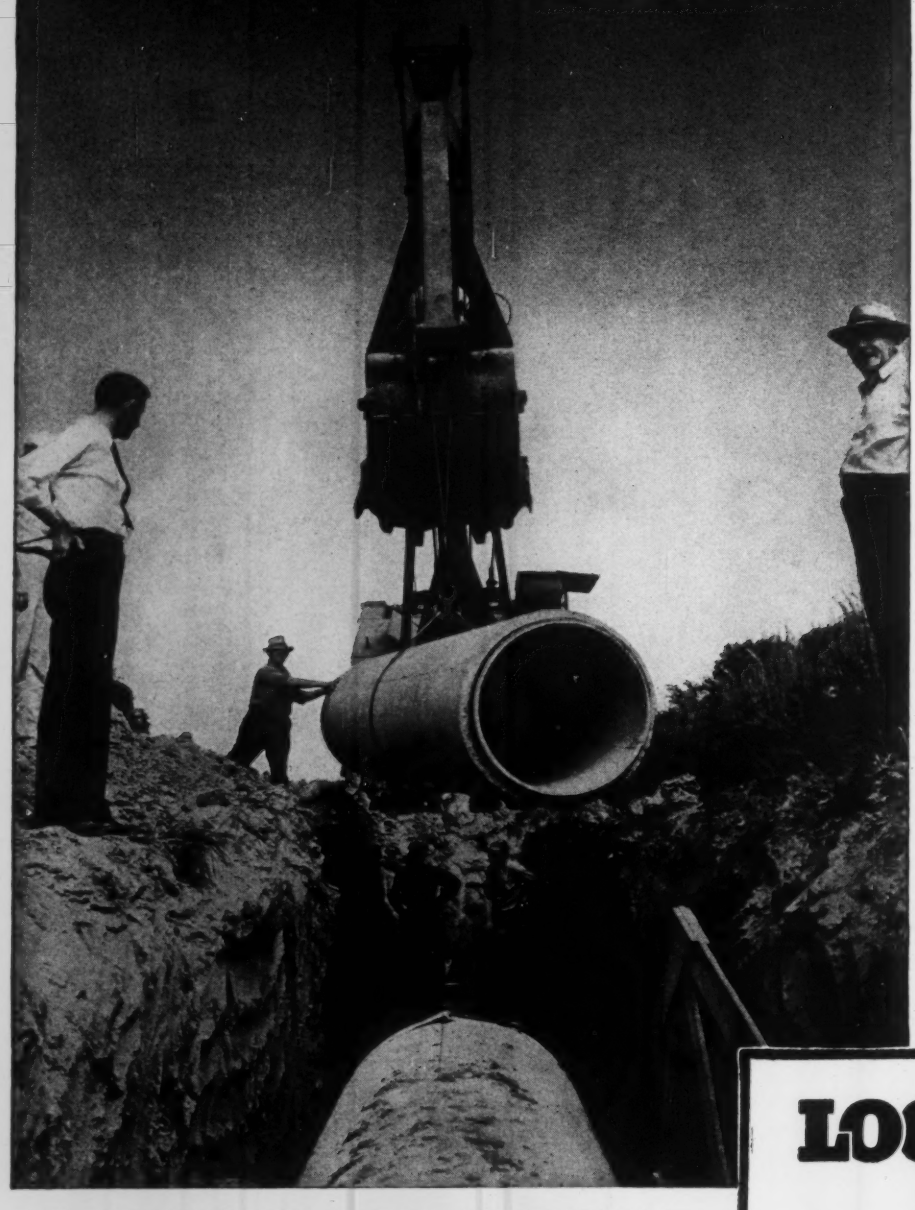
SPILLWAY CLOSURE BLOCKS

Cherokee Dam

In the vital Hampton Roads area....



● These water supply lines, totaling 82 miles of large diameter pipe, are part of seven Federal Works Agency water works projects installed at Hampton Roads, Virginia.

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Something to Think About

A Series of Reflective Comments Sponsored by the Committee on Publications

Taxation and Municipal Engineering

By EDWARD M. BEARDSLEE, ASSOC. M. AM. SOC. C.E.
CITY MANAGER, DE LAND, FLA.

ONE field of engineering on which the war will have a far-reaching effect is that of municipal engineering—not only as regards present restrictions on labor and materials, but also from the point of view of long-term financial considerations.

It may be that it is now too early to discuss the post-war world; discussion can be based only on assumptions that we will resume our normal pursuits, and that we will be permitted to pick up our civilian interests where we laid them down. Nevertheless, what we do after the war will depend in large measure on the extent to which we prepare now.

The many types of municipal engineering projects are financed either entirely by the municipality, through bond issues or reserve funds, current taxes, local assessments, or as in the recent past, by loans and grants from the federal government. During the years immediately following 1933, many municipal improvements were made possible by federal loans and grants, the municipality financing its share of the expense through bond issues. The public works programs during the past seven years have gone a long way toward educating engineers and public officials in project management and financing. At the same time those programs have consumed most of the financing power of the cities, owing to statutory or other limitations on their bonded indebtedness.

Planning and financing go hand in hand. While engineers know more about financing than they did a few years ago, they cannot see how many cities can do any more financing through bond issues for many years to come. For that reason the question of future public works is one which already is troubling many engineers. With so many outstanding bonds to be retired, and with municipal income growing less every month that the war continues, the question of providing funds for future works of expansion or maintenance is one that requires serious study.

The municipality is the foundation of our entire tax structure. Federal, state, and county taxes may form the superstructure, but the main revenue of all taxing agencies is derived from the highly improved property of the municipality, with its demand for utility services, its occupational and special licenses, its concentrated personal-property taxes, and its high-bracket incomes. If exorbitant taxes make it impossible for people to own homes, and the tax burden keeps business from operating at a profit, then the municipality has no source of revenue left—since taxes on vacant or ownerless property cannot produce adequate revenue. If there is no demand for real property, it has no resale value; where there is no resale value the tax value is artificial; and without tax value there can be no tax income. It follows that a tax can be confiscatory without being of benefit to the owner or to the taxing agency.

In many states the homestead exemption provisions throw the burden of operating or general-fund taxes on the businesss of the city. The home owner pays only a small tax to retire bonded indebtedness. Thus the urban business man must pay a license to his city, county, and state to do business; he also pays a real- and personal-property tax to the same agencies, not only on the business but also on any private property he may own. In addition there are the federal income tax, the various forms of federal business taxes, old-age benefits, and the like.

In the order of their power and ability to collect, the taxing agencies are: the federal government, the state, the county—and last, the municipality. In theory, the county and city have equal power, but the city is sometimes unfortunate in that its charter fails to designate how and by whom a levy may be collected, with the result that some taxes may be collected only by courtesy of the taxpayer. Whether and how the federal government can appropriate the right to collect taxes and other payments before other claimants is another unsettled question of states' rights. No such question arises with regard to the state and the county, which lay claim to taxes ahead of the municipality. The wealth of the city is built up by the efforts of its citizens, who are striving to secure the better things of life. If they are to enjoy a better environment, they must pay higher

municipal taxes; however, they have something in return for their money.

The instability of our entire tax structure might be illustrated by comparing it to an inverted cone resting on the municipality as a foundation and widening out to represent county, state, and federal taxation, the latter coming at the top, the widest part. Such a structure will have little stability at best but none at all if its foundation is defective. The superimposed burden of taxes from county and other agencies can depress real property values to the point where they will have no trade value.

In many cases excessive tax valuations have the same result, whether assessed by the city or by the county. Florida real property values, for example, are terribly depressed. Most of the assessments are based on values established during the so-called boom, and these values are no longer in the property. Urban properties listed on the tax rolls at more than a hundred dollars a front foot frequently will not yield as much as twenty dollars a front foot, and opportunities for sale are rare. Residential property is in the same condition, and few people are willing to pay three thousand dollars for a property that is assessed at fifteen thousand. Assuming on such a piece of property a homestead exemption of five thousand dollars and a levy of twenty-eight mills, the city taxes will be two hundred and eighty dollars. Frequently the property cannot be rented for this amount.

The result is that more and more property becomes idle—taxed, but not yielding a tax income. The downward spiraling of municipal income that must result from such a condition is a source of deep concern, not only to those who are responsible for the operation of the city, but also to the citizens themselves, who can see only ruin and chaos in the ever-widening gap between revenues and expenditures.

This condition has two effects on municipal engineering. First, the strictest economy is necessary to spread the income as thin as possible, to make it go as far as possible, and to serve the greatest number of people. Economy means reduction in salaries and personnel, with the usual likelihood of reduced efficiency. Utilities have to be operated with a minimum of maintenance and repairs, and major repairs must be delayed as long as possible.

Second, there are no funds available for improvements or expansion, and plans for anything but essentials have to be shelved. Strangely enough, the present emergency presents the paradox of requiring the maintenance of utilities at their present level, while accommodating population increases of from 25 to 100%. This is the picture and the problem facing the engineer today. It is not a hopeless situation, but it is one that only the engineer can solve satisfactorily.

Conditions today furnish an excellent laboratory for an analysis of community problems. Engineers engaged in war work have opportunities to observe in communities other than those in which they normally practice. This is especially true of men who have done camp and cantonment work.

Studies for future municipal public works must be made on the basis of need—not need as we have known it in the past, but actual need from the point of view of physical necessity. By this is not meant the desire of a group or administration to erect a monument—we have too many of those already. Based on a population survey, a city may think it needs increased sewerage and disposal facilities, and a larger water storage capacity. A critical analysis of the population increase, however, may reveal that it is transient in nature, or very temporary at best, and that a change in the method of operating the utilities will solve the problem. The engineer must be prepared to make such a critical analysis, and to support his recommendations.

On the other hand, it may be that a war industry started in a town may become a permanent plant, and may even require expansion in time of peace. In that case, consideration must be given to the need for increased water supply, sewerage, and even new residential areas. Here again the engineer must be ready to make recommendations and submit plans, so as to fill the need with the funds and the materials on hand. It takes no genius to get results with unlimited money and materials.

It is in the field of critical analysis and use study that the engineer can best serve during the war—that is, if he is not actively engaged in war work, and even then he must devote some time and thought to the future. He must visualize the top-heavy tax superstructure and keep before him the need for reducing the dimensions and the character of the load. Although the cone we have imagined is a typical form, this cone actually should be reversed so that the widest end rests on the municipality. This would lower the center of gravity, thus making the entire structure more stable.

The reduction of taxes should start at the top—with the federal government. At the same time subsidies and grants-in-aid should be controlled, and as far as possible eliminated altogether. Such a measure would tend to reduce the diameter of the top of the cone and its height. The next measure needed would be the spread of state and county taxes through an equalization of assessed values, thus giving the municipality the benefit of a wider distribution of assessed values.

If the levies of the taxing agencies above the municipality are reduced, there will be fewer municipal tax delinquencies. If a tax levy is based on collecting 75% of the assessment, and conditions change so that collections actually amount to 85%, a distinct benefit can accrue both to the city and to its people. The result will be greater revenue to the municipality with no increase in either millage or assessment.

Again, in such a step the engineer must be the prime mover. Municipal improvements have been accomplished in the past without the aid of federal funds. Similar improvements can be accomplished in the future, provided the improvement is needed to supply an actual physical lack, and not merely to provide work for the idle. Such social reform should be accomplished in some other way.

The engineer must have the conviction and the courage to follow his logical conclusions after a thorough study of the situation. When he has done that, he will have served his country and his profession as it was his duty to do.

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VOLUME 13

NUMBER 8

Manhattan Prepares for Postwar Construction

Completed Plans for Highways and Sewers Prepared for Emergency Use, if Necessary

By DEAN G. EDWARDS, M. AM. SOC. C.E.
CHIEF ENGINEER, BOROUGH OF MANHATTAN, NEW YORK, N.Y.

THEORIES about postwar engineering are plentiful; practical plans are correspondingly scarce. When the showdown comes and public works are required at once, attempts to improvise plans, to make them materialize out of thin air will be futile. Only completed working drawings and specifications, ready for immediate use, will avail. With commendable foresight a number of such projects have been developed by the Borough of Manhattan. It is to be hoped that Mr. Edwards' description of this prevision will inspire other engineers to like endeavors.

PREPARATION of plans and specifications for postwar construction by the Engineering Department of the Borough President of Manhattan has been in progress since early in 1942. The projects for which construction drawings are being made were a part of our construction program before the war and were not originated for the purpose of taking up the slack of employment after the war or to provide work for designers and draftsmen during the war. However, proceeding now with the preparation of these plans and specifications does have the effect of providing a ready reservoir of extremely useful employment for men returning to civilian life after the war.

Actually, many of these projects would now be in construction had not the war intervened. With war activities drastically curtailing purely civil construction, over $1,700,000 was made available to us by the Board of Estimate of New York City for preliminary studies and for the preparation of contract drawings for about $64,000,000 worth of construction.

Designs have been begun for a variety of projects—some of which are indicated roughly on Fig. 1. These include about twenty-three sewers (totaling about 9 miles) throughout the Borough of Manhattan, which must be enlarged or rebuilt at an estimated cost of $5,000,000 because of changed conditions; three Central Park transverse roads, entrances and exits of which will be relocated to provide greater traffic efficiency and safety; and extension and completion of the express waterfront highways, which will ultimately encircle the Borough.

For certain projects studies only are being made. These projects include several cross-town express highways to relieve the present surface congestion; bus terminal approaches; and the rehabilitation of some areas, which will entail alterations in the present street system.

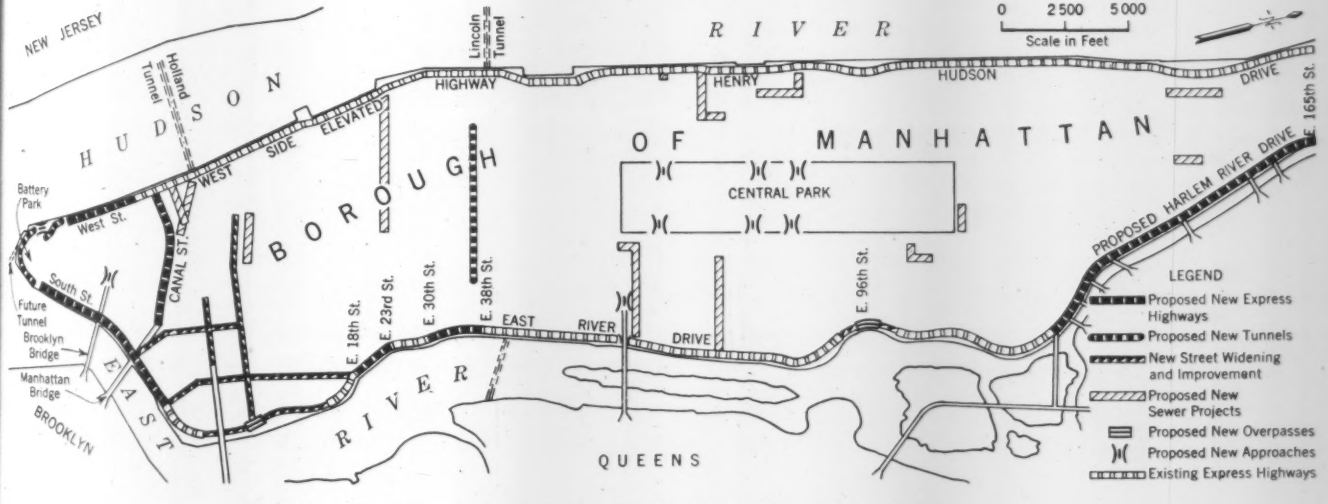
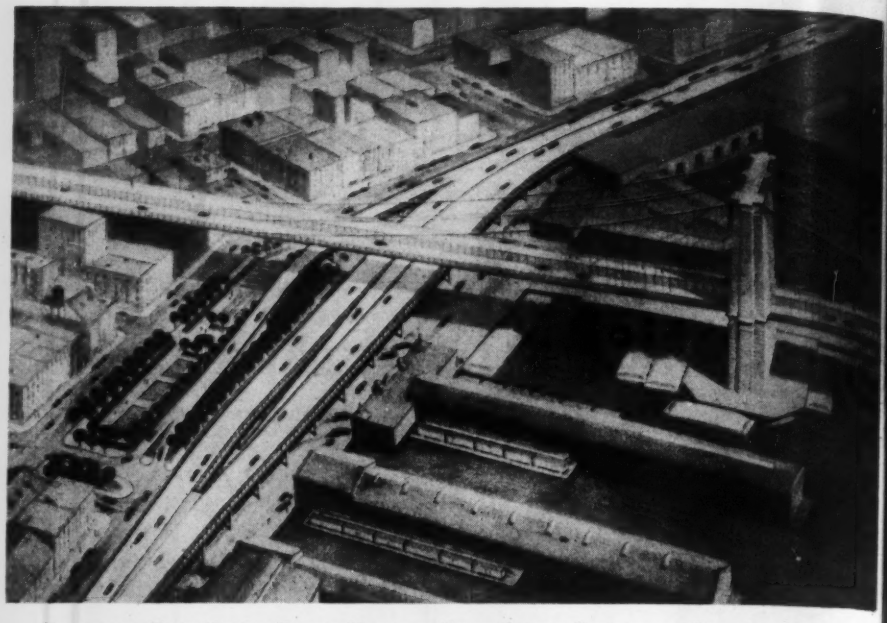


FIG. 1. PROPOSED POSTWAR WORKS PROGRAM—BOROUGH OF MANHATTAN
Sewer and Highway Connections Only Are Planned in the Omitted Part, North of 165th Street

Plans and specifications are completed for the extension of the West Side Elevated Highway from its present end at Duane Street to the vicinity of the Brooklyn-Battery Tunnel Plaza (Battery Park). This will be a six-lane steel and concrete structure about 3,800 ft long and costing about $3,000,000. All preliminary engineering work has been done, so that the contract could be let on a few days' notice.

As a part of the city's contribution to the Brooklyn-Battery Tunnel, a four-lane underpass beneath Battery Park (Fig. 1) and a two-lane tunnel approach from the West Side Elevated Highway to the Tunnel Plaza will be built by the Borough President of Manhattan. Surveys and borings have been completed. As of June 1943 the structures were more than 80% designed and by December 1943 this project should be ready for letting. The Battery Park Underpass will cost about $3,500,000, and the plaza approach about $500,000.



PROPOSED EAST RIVER DRIVE VIADUCT OVER SOUTH STREET, AT BROOKLYN BRIDGE

Some portions of the East River Drive were not completed before the war. At present, South Street is used for East River Drive traffic between Montgomery Street and Battery Park (Fig. 1). A viaduct about 8,500 ft long will be built in South Street to carry East River Drive traffic over South Street. Preliminary surveys have been in progress, to be followed by design work. This structure will cost about $5,500,000.

The commercial marginal street is used temporarily for East River Drive traffic between 18th and 23d streets and between 30th and 38th streets. Viaducts will be built to separate the heavy commercial cross traffic from the through passenger car traffic. The viaduct between 18th and 23d Streets will have six lanes, with exit and entrance ramps south of 18th Street and north of 23d Street. Borings and surveys have been completed, and the design work is well over half finished. Within a few weeks, this project would be ready for letting. Cost of construction will be around $2,000,000.

Likewise, the East River Drive will be elevated over the marginal street between 30th and 38th Streets, with exit and entrance ramps to the north and south. This viaduct has not yet reached the design stage but will follow the others in due course. The construction cost will be about $2,250,000.

A major link in the waterfront highway will be the Harlem River Drive, extending north from the Triborough Bridge and connecting with the Harlem River Speedway north of 165th Street. Borings, preliminary surveys, and layout have been completed, and the southerly section is now (August 1943) in the process of design. The cost of this project will be in excess of $12,000,000.

A number of other highway projects have been advanced to such a point that construction could be started on short notice. These include widening certain streets to provide continuous commercial highways, extending other streets, eliminating grade crossings, and constructing service streets so that commercial traffic may be excluded from express highways. Various specific projects are shown by location on Fig. 1, but not identified by name.

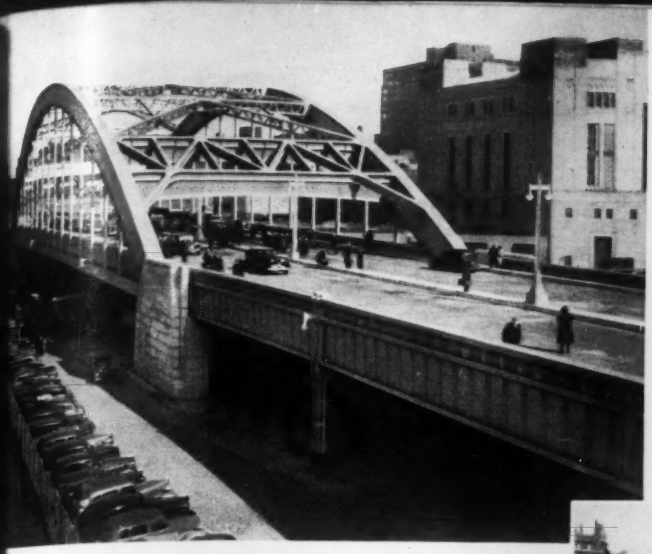
Also included in the East River Drive program are several pedestrian bridges for access to the park area along the river front north of 92d Street. Near the south end of the Drive similar bridges will be built in conjunction with a local lane paralleling the existing roadways.

In preparing contract drawings, special consideration has been given to possible changes in construction techniques and to materials which may become available after the war. For example, it is planned to invite bids on completely welded steel structures for the viaducts as an alternate to completely riveted structures as heretofore designed. Later there may be other new ideas to incorporate in our plans.

It will be seen that the Borough of Manhattan has taken extended and practical steps to be ready for a postwar emergency. Speaking as of the summer of 1943, the status of these projects is as follows: The design and specifications for the West Side Elevated Highway extension are complete; the drawings for the Battery Underpass and Approaches are over 80% completed; and the designs of the East River Drive structure from 18th to 23d streets and of the East River Drive Overpass at 96th Street are about three-quarters completed. Other highway projects, including Central Park transverse roads and the Harlem River Drive, are about 10% designed. In general, the sewer projects are over 30% designed, and four of them are completely designed.

The personnel engaged on this work is the Engineering Department of the Borough President. Although severely depleted by enlistments in the armed services and by losses to defense industries, the Department still has a sufficient staff to prepare a postwar program of construction, a large part of it to be available immediately after the war. Outside consulting services have been used for architectural and tunnel ventilation designing.

These projects are a part of the Mayor's Postwar Construction Program. They are being developed under the supervision of Borough President Edgar J. Nathan, Jr., and Commissioner of Borough Works, Walter D. Binger, M. Am. Soc. C.E. The writer is Chief Engineer for the Borough.



ARCH STRUCTURE CARRIES WEST SIDE DRIVE OVER CANAL STREET. PHOTO AT UPPER RIGHT. EAST RIVER DRIVE NORTHWARD FROM 121ST STREET APPROACH AT LEFT TO TRIBOROUGH BRIDGE

CLOSE-UP OF GRADE SEPARATION AT 48TH STREET AND THE EAST RIVER DRIVE, LOOKING NORTH

GRADE SEPARATION PERMITS ACCESS TO EAST RIVER DRIVE AT 48TH STREET
Queensboro Bridge (59th Street) in Background

"BEFORE AND AFTER" VIEWS OF EAST RIVER DRIVE, 54TH TO 57TH STREETS LOOKING SOUTH
Constricted Conditions Dictated Unique Double-Deck Highway. Public and Private Recreation Areas on Top

Concrete Grade Separation at South Charleston, W.Va.

By L. L. JEMISON, M. AM. SOC. C.E.
BRIDGE ENGINEER, WEST VIRGINIA STATE ROAD COMMISSION, CHARLESTON, W.VA.

EXCESSIVE interruption of highway traffic to and from South Charleston industrial plants at a railroad crossing dictated the construction of a grade separation. Location, design, and methods of construction were all limited by the restricted right of way available. The rerouting of through traffic during construction, as described by Mr. Jemison, was so planned as to leave the contractor the most possible room for his work on the reinforced concrete structure.

ALTHOUGH in general few grade eliminations are being built because of priority problems, one large project of this type has just been completed in South Charleston, W.Va. The comparatively large quantities of reinforcing steel required could be released because of the value of the crossing in expediting both rail and highway traffic serving this busy industrial area.

The South Charleston Overhead is located on MacCorkle Avenue on the Charleston–Huntington road, U.S. Highway Route 60. This location (Fig. 1) is on the designated strategic network of highways and also covers a portion of the access road which was certified by the U.S. Navy Department as important to the war effort, serving plants in South Charleston and in the Kanawha Valley.

The overhead eliminates a grade crossing of a heavily traveled industrial line of the Chesapeake and Ohio Railroad Company. A 24-hour railroad traffic count, taken at the crossing by the State War Transport Division about one year before construction began on the overhead, listed 28 freight train crossings and 48 other crossings, including switching and making up of freight trains, giving a total of 76 interruptions of highway traffic, blocking a total of 1,276 vehicles. The highway was blocked about 1 hour and 15 minutes during the 24-hour study. At the time construction began on the separation structure, it was estimated that the train movement over this service track had increased approximately 10% owing to increased production at the plants served.

Highway traffic, before construction of the overhead, was estimated to be 20,000 vehicles during an average 24-hour period. The greater part of this traffic is to and from the Kanawha Valley.

Previous to construction of the grade separation, the highway had two 10-ft lanes of concrete pavement, constructed about 8 years ago and inadequate for the present heavy traffic. Within the preceding year, a new 4-lane highway was practically completed from the Patrick Street Bridge over the Kanawha River at the west end of South Charleston, to St. Albans, a distance of 10 miles. The crossing is on the 4-lane route, U.S. 60.

The right of way for the overhead, which is located about 0.8 mile west of the Patrick Street Bridge, was restricted by the property line of an industrial corporation on the north side, and the Chesapeake and Ohio Railroad on the south side. To detour heavy traffic during construction, an easement was obtained from the Railroad Company to construct and maintain a one-lane road for westbound traffic. This road could not be constructed until the south wings of both abutments were completed; therefore it was specified in the contract that the traffic be maintained on the original paved highway until the south wings were completed. The

ABOVE: CROSSING BEFORE GRADE SEPARATION WAS UNDERTAKEN

RIGHT: GRADE SEPARATION WHICH IS NEARING COMPLETION

one-lane paved detour ran along the south side of the structure. This lane was just north of the main line of the Chesapeake and Ohio Railroad, which parallels the center line of the bridge.

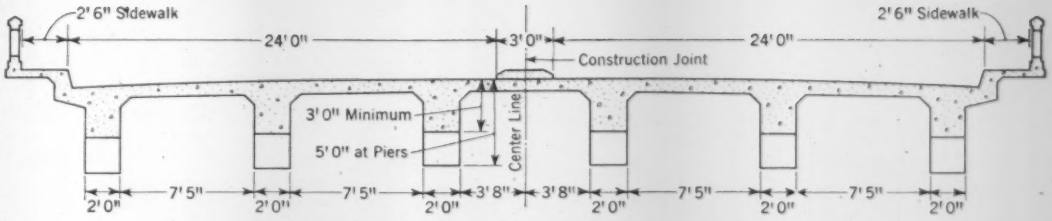


FIG. 2. SECTION THROUGH THE CONTINUOUS DECK SPANS

Eastbound traffic was detoured from U.S. 60 at the west end of the structure over a paved road through an underpass to the Kanawha and James River Turnpike, on the south side of the railroad. Traffic crossed over the railroad grade crossing returning to U.S. 60 at the Patrick Street Bridge. The increased distance for main eastbound traffic along the detour was 0.95 mile. It was possible for eastbound traffic to avoid the grade crossing to the Patrick Street Bridge by continuing on the turnpike underpassing the railroad to cross the Kanawha River at the South Side or Kanawha City Bridge. On this latter route there was no appreciable increase in the distance for eastbound traffic on U.S. 60.

The Overhead structure, of reinforced concrete, consists of continuous deck girders and simple-slab spans. The over-all width of the superstructure is 58 ft, which provides two 24-ft, two-lane roadways, a 3-ft median strip, and two $2^1/_2$-ft safety walks.

The structure which is 1,000 ft long, consists of six continuous three-span units, eight trapezoidal simple-slab spans, and one simple-slab span over the railroad track. The continuous spans have six girders spaced about $9^1/_2$ ft from center to center, with a minimum depth from top of slab of 3 ft and a maximum depth at the piers of 5 ft, as shown in Fig. 2. The piers consist of three reinforced columns on independent footings with a reinforced concrete cap, except the piers on each side of the railroad track, which have eight reinforced concrete columns on a common footing. All piers and abutment footings are supported on creosoted timber piles. The abutments have U-shaped wing walls, 240 ft long on the east end and 210 ft long on the west end, to retain the fill of the approach ramps. A gravel fill was placed between the wing walls at both approaches. The gravel fill was used, so there would be a minimum settlement of fill.

Sixteen borings were taken to determine the character of the subsoil, and the data indicated a formation of cinders, loam, and clay for an average depth of about 18 ft below the surface, then a formation of sand or gravel to an average depth of about 38 ft. Below this, hard shale or sandstone was found. Near the east abutment, the shale or sandstone strata were from 10 to 15 ft higher. To develop adequate bearing strength, the piles should penetrate the sand and gravel several feet, and it was questionable whether timber piles could be driven in this formation a sufficient depth. Before the contract was let, three untreated timber piles were driven to a depth of about 35 ft. At this depth a bearing of 25 to 30 tons was indicated by the application of the Engineering News formula. The design was based on 18 tons per pile.

The height of the structure was governed by the clearance required by the railroad. Overhead was placed on a vertical curve for the entire distance between abutments of 1,000 ft, based on a 5% grade at each end. The top of the wings, and the railing on the wings to conform with the highway profile, are on a concave vertical curve starting at the abutments. Approach from the east is on a 1° curve ending 185.5 ft west of the east abutment.

The railroad siding leading to the industrial plants, as shown in Fig. 3, enters under the overhead on a 15° curve to the right, and beginning at the intersection of the highway center line, it leaves on a 15° curve to the left.

It is noted in Fig. 3 that at the intersection angle of the center lines of the railroad and the highway, the right-angle distance from center to center of piers is 24 ft, while at the end of these piers, the center-to-center distance parallel with the center line is 69.13 ft. The railroad required a vertical clearance of 22 ft and a clearance, from center line of track, of 10 ft, making a span of 20 ft at the point of intersection, which increased on each side of the center line of the overhead to a maximum span of $21^1/_2$ ft. In order to have a minimum grade on the structure, a simple slab 19 in. in depth was used for the span over the railroad. To support the skew side of the slab along the curves at the sides of the piers, two 24-in. 150-lb beams were used. These had two 16-in. by $^{11}/_{16}$-in. cover plates on each flange with diaphragms between beams, and anchor bars through the webs of the beams into the concrete slab.

The beams are directly under the sidewalk and the bottom of the concrete slab is flush with the bottom of the beams. This was so arranged that the sidewalk over the crossing would line up with the sidewalk of the simple spans on each side of the railroad span. This arrangement of beams permitted use of the same rail section on the railroad span as on the rest of the structure.

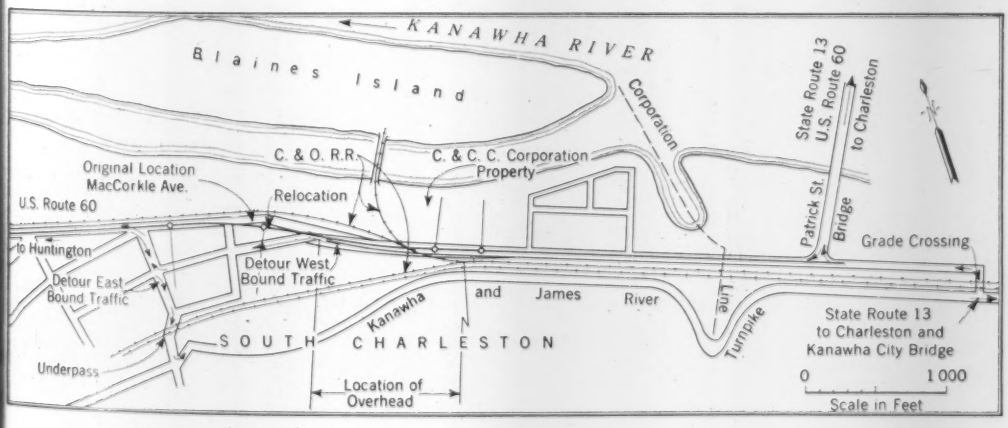
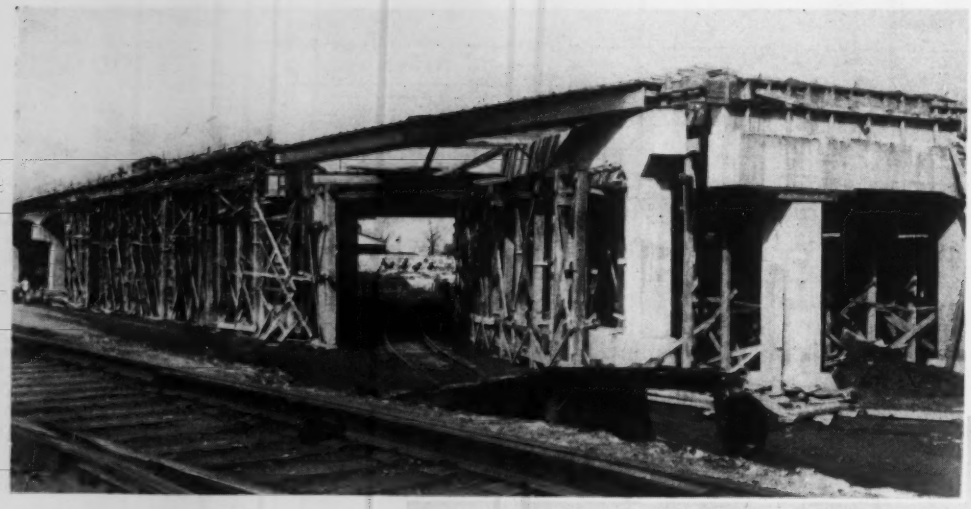


FIG. 1. SITE PLAN OF THE SOUTH CHARLESTON "OVERHEAD"

In the 3-span continuous units, most of the negative longitudinal reinforcement was concentrated in the stem of the girders over the intermediate supports, and some additional longitudinal reinforcement was placed in the top of the slab



STEEL BEAMS SUPPORT SLAB EDGE AT SKEW CROSSING OVER RAILROAD SPUR

between girders at the intermediate support. The girders were fixed at one intermediate pier. Cast-steel rockers for expansion were provided at both ends of the unit, and to save critical material, malleable iron plates were used for expansion at the other pier.

A longitudinal construction joint was used at the center line of the roadway so that the contractor could place half of the roadway section in the three-span unit in one continuous pouring. Approximately 190 cu yd could thus be poured at one time. A construction joint was provided in the curb about 3 in. above the roadway slab so the sidewalk slab could be poured separately.

SIMULTANEOUS CONCRETING REQUIRED FOR CONTINUOUS SECTIONS

Transit-mixed concrete was used. The batching plant was a mile and a quarter away. Vibrators were used in placing all concrete. The contractor was required to pour the continuous sections at four points simultaneously, beginning at the center of the middle span and working toward each end, and also beginning at each end and working toward the center, until the girders were completed to the bottom of the roadway slab. Then the roadway slab was poured, beginning at each end and at the center of the middle span, and working toward the two piers. A minimum of 25 cu yd of concrete was required to be placed per hour. Each one of these half units was poured in about 8 hours.

Because of the restricted right of way, for part of the structure the contractor constructed a ramp for the movement of the transit mixer along the half section of roadway that was poured first. In placing the concrete for the other half, the transit mixer traveled on the section that had been poured. In pouring other parts of the superstructure, a crane lifted the concrete from the transit mixer.

During cold weather the concrete was protected by housing along the sides and top, and heat was applied beneath the forms.

On May 4, 1942, the contract was let to the Pocahontas Construction Company, Lewisburg, W.Va., and Monty Brothers, St. Albans, W.Va. The work was completed in June 1943. The total contract bid was $398,539 including approximately $10,000 for constructing and maintaining the one-lane road for westbound traffic. About 1,200 creosoted timber piles were used, with an average penetration of about 28 ft. Approximately 10% cutoffs were permitted, and for cutoffs over 2 ft long an allowance of 50% of the unit price was made.

PLAIN CONCRETE SURFACES REDUCED FORM WORK TO A MINIMUM

The project was financed by the West Virginia State Road Commission using federal aid grade crossing and access road funds. The Public Roads Administration was represented by William Brewster, Senior Highway Engineer, and J. C. Cobb, Highway Engineer.

The structure was designed by, and the construction was under the supervision of, the West Virginia State Road Commi sion, for which M. L. O'Neale, is Chief Engineer; Geo. W. McAlpin, State Construction Engineer; L. J. Jemison, Bridge Engineer; Charles Boggs, Resident Engineer; and Paul McDowell, Inspector.

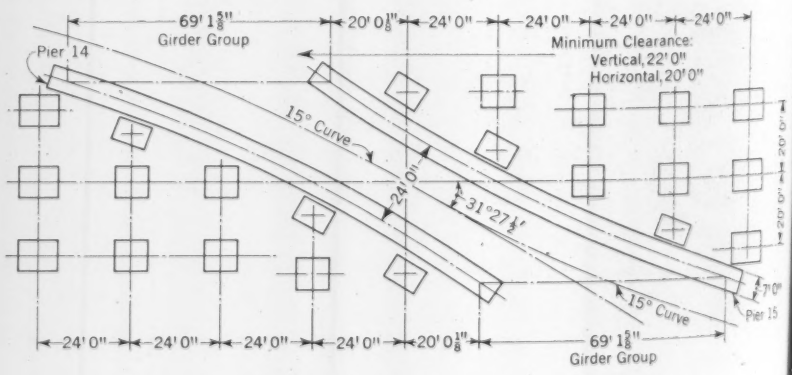


FIG. 3. FOUNDATION PLAN OF THE SKEW CROSSING

Flow Diversion at Niagara Falls

II. Hydraulic Aspects of the Remedial Weir

By C. G. Cline

Senior Assistant Engineer, Dominion Water and Power Bureau, Niagara Falls, Ontario

PRESERVATION of the beauty of the Falls was the first consideration of the Special International Niagara Board in planning a remedial weir in the cascades above Niagara Falls. At the present time, however, construction of the weir can be justified only by the increased power made available for war use. The hydraulic properties of the weir have been and are continuing to be studied closely by Mr. Cline, as reported in this paper, which is second in a series begun in the July issue. Mr. Cline's paper was originally presented at the Niagara Falls Meeting of the Society.

As a result of the natural regulation provided by Lake Erie and the upper Great Lakes, the flow of the Niagara River is remarkably steady and does not have the great fluctuations common to most streams. The discharge of the river is controlled primarily by the elevation of Lake Erie at its outlet at Buffalo, which is subject to change from day to day, from season to season, and from year to year. Slow changes in the mean elevation of the whole lake result from differences between the run-off into the lake and the outflow. More rapid changes at any one point may result from oscillations of the lake surface caused by wind or by differences in barometric pressure. Usually these changes are small, though occasionally a severe storm may cause a rise or fall of several feet at the lake outlet within a few hours. All fluctuations in lake level at Buffalo, whether slow or fast, cause corresponding fluctuations in the stage and discharge of the river.

Between Lake Erie and the Falls, there are 22 miles of unbroken water followed by 3,500 ft of cascades that form the approach to the Falls themselves. The flow from Lake Erie is controlled by a limestone ledge at the outlet of the lake, Fig. 1. In the first four miles, there is a fall of 5 ft with relatively high velocities; an additional 5 ft is distributed over the remaining 18 miles with much lower velocities. Under these conditions, the control at the lake outlet acts as a submerged weir, the discharge being controlled primarily by the headwater elevation of Lake Erie at Buffalo and, in the second place, by the tailwater elevation below Black Rock. Thus it follows that the discharge from the lake, and ultimately the level of the lake itself, are affected slightly by anything that interferes with normal flow conditions at any point between the lake and the cascades.

Several sets of meterings of the river discharge have been made at various times during the past 50 years at, or near, the Fort Erie-Black Rock railway bridge by field parties of the U.S. Lake Survey and the Dominion Water and Power Bureau. The most recent meterings were made by the Lake Survey in 1931. Following this work, previous discharge equations for the Niagara River were reviewed by the Lake Survey, and a new equation was derived which includes a term for the fall between the Buffalo and Black Rock gages as follows:

$$\text{Discharge} = 1{,}989\ (\text{Buffalo} - 556.78)^{3/2}\ (\text{Buffalo} - \text{Black Rock})^{0.3} \quad \ldots\ldots (1)$$

Coming downstream, the river is divided into two channels by Grand Island, about 60% of the flow passing down the west channel.

Below Grand Island, the wide portion of the river extending for some three miles to the head of the cascades is known as the "Chippawa-Grass Island pool," though the designation "pool" is somewhat of a misnomer since the slope in this portion of the river is considerably steeper than in the channels above. From this pool, Fig. 2, water is diverted at Chippawa, on the Canadian side of the river, for the Queenston power plant; also near Grass Island, on the United States side, for the two plants of the Niagara Falls Power Company. The upper

Cableway in Place for Construction of Remedial Weir

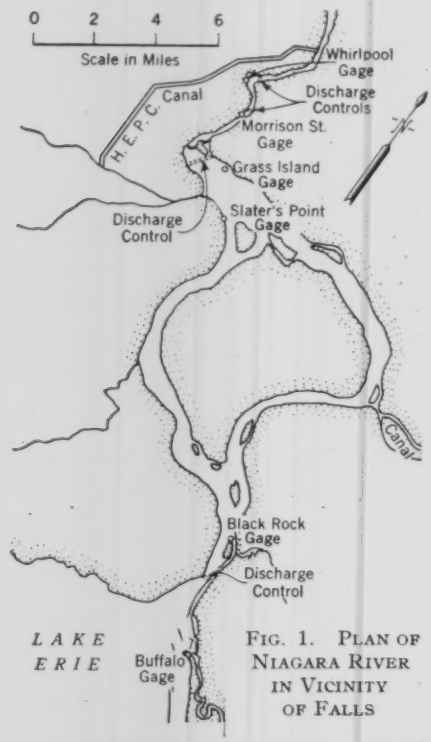


FIG. 1. PLAN OF NIAGARA RIVER IN VICINITY OF FALLS

crest of the cascades, which forms the control for this pool, is divided into two parts by Goat Island, 95% of the river flow passing south of the island and over the Horseshoe Falls and the remaining 5% passing north of the island and over the American Falls. From the upper end of Goat Island, a rock shoal extends upstream for 3,000 ft or more, and the water for the American Falls passes north of this shoal.

Through the cascades and Horseshoe rapids, there is a fall of 50 ft, each line of white water forming an independent control that prevents any change in water level from being transmitted upstream. In the rapids above the Horseshoe Falls, much of the water is carried by two main channels which tend to converge at the crest of the Falls. Farther upstream, the two channels are separated by an extensive area of shoal water in mid-river.

Below the Falls, the water level in the Maid-of-the-Mist pool is controlled by the solid rock stream bed at the head of the Whirlpool Rapids. Through the rapids, there is a fall of 45 ft in a distance of one mile. There is a similar control at the outlet from the Whirlpool with a fall of 45 ft below it in a distance of 4 miles. The remaining 7 miles of river has a comparatively flat gradient to Lake Ontario.

The new weir in the upper Niagara River is part of the remedial works recommended by the Special International Niagara Board. Its general purpose is to raise the level of the water in the Chippawa-Grass Island pool. At low water, a rise of several feet might be permissible, but too great a rise at high water might cause serious damage by overtopping the parapet along the canal of the Niagara Falls Power Company. The allowable amount of rise at low water was specified as 1 ft at the Grass Island gage, which is located near the power company's intakes. Because of the hydraulic characteristics of the submerged weir, the rise at high water will be somewhat less than 1 ft.

The submerged weir is being built in the main channel of the Niagara River about one-half mile upstream from the crest of the first main cascade. From the information originally available, this location was selected as being most advantageous for the purpose desired. The suitability of the adopted location was confirmed when, following the erection of the cableway, reliable soundings and velocity observations were obtained from the cable car. The bottom of the river is flat bedrock with occasional crevices and some boulders. The cross section is comparatively uniform (Fig. 3) with a maximum depth of 15 ft at the present stage of the river. The maximum velocity observed was 8 ft per sec, or $5\frac{1}{2}$ miles per hour. The broken line outlines the weir.

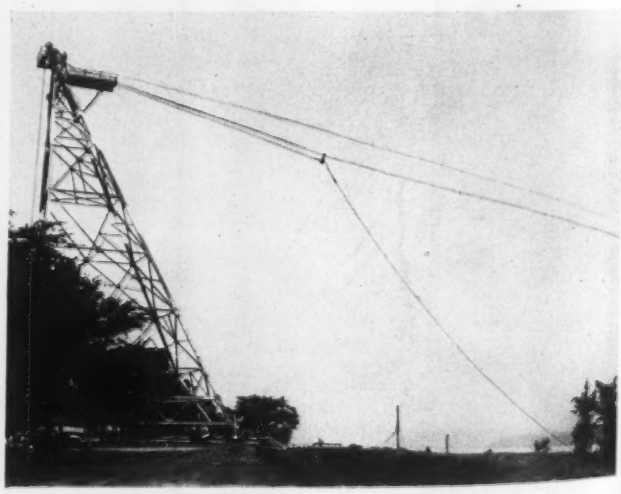
The elevation tentatively selected for the crest of the weir will give an overflow depth of 6.5 ft at low water, which should be ample for passing ice. At the Canadian end of the weir, a gap several hundred feet long is to be left between the weir and the shore in order to maintain present water levels at the intakes of the power plants farther downstream. However, both the elevation of the crest and length of weir were still subject to minor changes according to the effect on water levels observed as construction proceeded.

Water levels in the Niagara River will be affected by the remedial weir from the crest of the Falls upstream to Lake Erie. The rise of 1 ft at Grass Island will decrease gradually as the distance upstream increases, the expected rise at Lake Erie being about one-tenth of a foot, which will be of some benefit to navigation.

In gage relation formulas, the problem is to devise a method of computing the effect of the weir on river levels as construction proceeds and thus determine the final elevation of crest and length of weir that will be required to give the desired rise in water level. Water-level recording gages have been maintained for many years at numerous sites along the river, so that a great number of precise water-level data are available for hydraulic studies. The construction of the remedial weir, which affects the water level at gages in the upper river, has no effect at gages in the lower river. Hence, the records of water level at these lower river gages afford a reliable basis of comparison for determining the effect of the weir at gages in the upper river.

The Morrison Street gage in the Maid-of-the-Mist pool near the head of the Whirlpool Rapids was adopted as the standard reference gage for these gage relation studies. This gage was established in 1922, and an excellent record is available as it is checked by comparison with the 3A gage of the Niagara Falls Power Company one mile upstream and also with a gage at the Whirlpool. The site is very sensitive to changes in river discharge, and its permanent discharge control is not affected by weed growth in summer and only rarely by ice in winter.

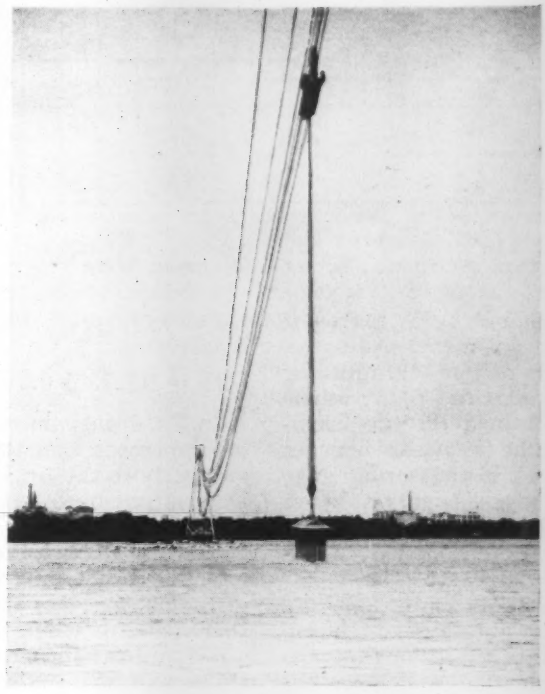
This gage is used to record the discharge of the lower river, an accurate rating having been developed by transferring downstream the rating of the Buffalo gage. The discharge formula, given below, was developed by the U.S. Lake Survey, following the meterings made in 1931, and was checked and adopted by the Dominion Water and Power Bureau. Meterings made from the construc-



TOWER ON CANADIAN SHORE
Track Cable $2\frac{1}{2}$ In. in Diameter Being Pulled Across the River

tion cableway in September 1942, with the addition of the discharge over the American Falls and the diversion made by the Niagara Falls Power Company, gave a close check on the accuracy of this formula. The Morrison Street discharge does not include the water diverted by the Queenston power plant, which must be added to give the total flow of the river, such as is given directly by Eq. 1. The formula for the discharge past the Morrison Street gage is as follows:

$$\text{Discharge} = 786\,(\text{Morrison} - 301.10)^{3/2} \quad \ldots\ldots (2)$$



INSPECTOR'S CAR ON CABLEWAY
Tower in Background Erected on Artificial Island

In developing gage relation equations between Morrison Street and any given gage in the upper river, the factors involved are the fluctuations in the river discharge and in the power diversions and the effect of the seasonal cycle. The fluctuations in river discharge are shown by the changes in the Morrison Street gage height, and the corresponding effect on the upper river gage is determined by means of what has been designated the "discharge coefficient." Similarly, "diversion coefficients" are used to show the effect of changes in the power diversions. The formulas used are of the following form:

$$G.H. = K + a(M - 336) - bQn - c\text{U.S.} \quad \ldots\ldots (3)$$

where $G.H.$ = computed gage height at given gaging point in upper river

K = elevation at given gage corresponding to elevation 336.0 at Morrison Street when diversion is zero. The numerical value of K varies during the open water season due to the effect of the seasonal cycle, as explained below

a = dimensionless discharge coefficient, indicating the amount of change at the upper river gage corresponding to a change of 1 ft at Morrison Street

M = mean daily gage height at Morrison Street gage

b and c = dimensional diversion coefficients for the Queenston (Qn) and United States (U.S.) diversions, respectively, showing the amount of lowering in water level at upper river gage for each 10,000 cu ft per sec of diversion.

In order to be able to use Eq. 3 for comparing the water levels obtaining at any given gage at various times, it is necessary to find the correct numerical values of the four unknowns: K, a, b, and c. The data available for this purpose consist of the mean daily gage heights, as recorded over a period of years at the Morrison Street gage and at the given upper river gage, and the mean daily power diversions as recorded at the various power plants. The problem is somewhat similar to an analysis of the tides, in which it is necessary to assign values for the effect of each of the various solar and lunar components. The mathematical process applicable in that case, known as the Method of Least Squares, can be used in this case also.

Using the mean daily figures, each day's record of gage heights and diversions will give the data necessary for one observation equation of the general form of Eq. 3, in which the mean daily gage heights and diversions are known and K, a, b, and c are the unknowns. From the selected number of observation equations, the corresponding normal equations are formed in the usual way. As explained in textbooks on the subject, the solution of the normal equations as simultaneous algebraic equations will give the most probable set of values for the unknowns—the set that will make the sum of the squares of the residuals a minimum. In this case, the residuals are the differences between the gage heights actually observed at the given gage and those calculated by using Eq. 3 with the set of values found for the unknowns.

The seasonal cycle, apparently as a result of the growth of aquatic plants, causes a rise in water level during the early summer, this rise usually reaching a maximum during July or August and then gradually decreasing to zero late in the fall. The general trend of the cycle is fairly consistent, though there are minor variations from year to year in its amplitude and time of incidence. In the Chippawa-Grass Island pool, the average maximum value is two-tenths of a foot. Below the crest of the cascades, no seasonal cycle can be detected.

In solving the normal equations for gages in the Chip-

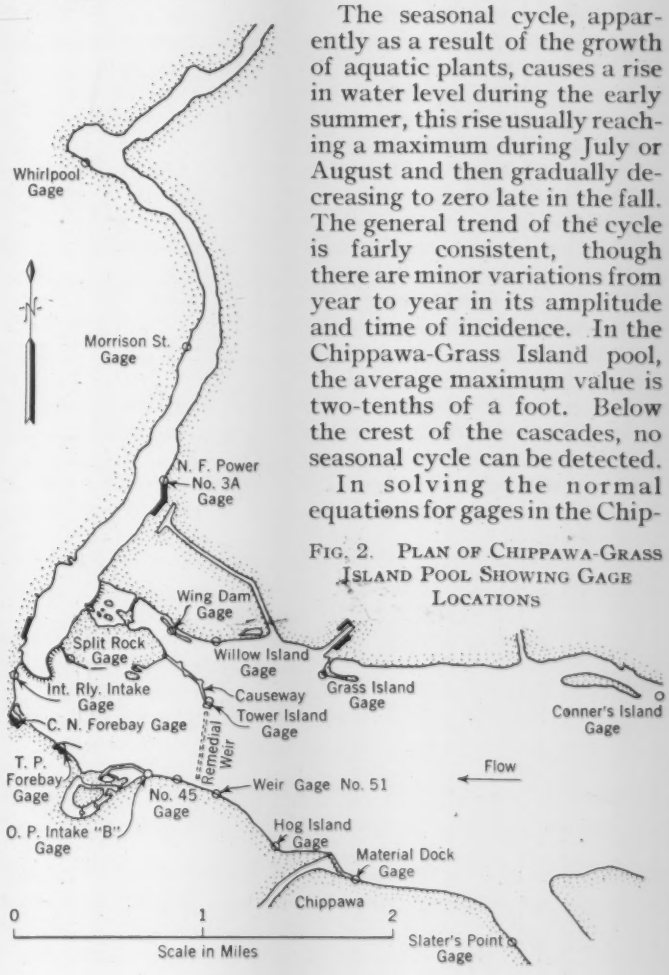


FIG. 2. PLAN OF CHIPPAWA-GRASS ISLAND POOL SHOWING GAGE LOCATIONS

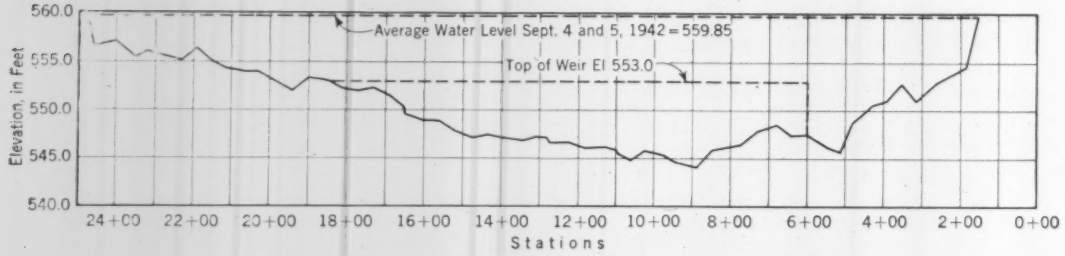


FIG. 3. RIVER BOTTOM PROFILE AT SITE OF REMEDIAL WEIR

pawa-Grass Island pool, this seasonal cycle makes it necessary to resort to an expedient not mentioned in the textbooks. Each month, or group of months, must be allowed to have its own independent value of K; otherwise incorrect values may be obtained for the coefficients. A similar procedure should be followed whenever there has been any permanent break in the normal gage relations. Usually the values of the coefficients only are derived by this method, the particular value of K to be used for any given purpose being determined by a different method, as explained below.

The Method of Least Squares has its limitations and must be used with discretion in computing gage relation formulas. It can determine accurate values of the various discharge and diversion coefficients only if there has been a considerable range of independent variation in river discharge and in each diversion during the period covered by the observation equations. The best results are obtained when there are accurate records extending over a period of several years, because then there will be sufficient variation in river discharge for the accurate determination of the discharge coefficient.

A similar opportunity for determining the diversion coefficients is obtained by using periods in which changes have occurred in the power diversions. If the available period of record is short, somewhat more reliable results can be obtained by discarding days in which the normal gage relationship has been disturbed by storms or other causes; the size of the residuals obtained from a preliminary solution is used as the criterion for discarding doubtful days for the final solution. In any case, all data used should be selected from the open-water season only, when there is no danger of ice disturbances.

The Grass Island gage, located at Grass Island near the intakes on the United States side of the river, is to be used as the main reference gage for determining the rise in water level that will result from the construction of the remedial weir. The weir, in accordance with the plan submitted by the Board, is to raise the water at this gage 1 ft at standard low water, the stage being specified because the effect of the weir will be somewhat less at high than at low stages. In 1926, standard low water was defined on the Lake Survey charts as 570.0 at Buffalo and 560.6 at Grass Island, which corresponds to a discharge of 155,000 cu ft per sec from Lake Erie. At that time, the total diversion from the Chippawa-Grass Island pool was about 31,000 cu ft per sec, so that the discharge over the crest of the cascades, the controlling factor on pool levels, was 124,000 cu ft per sec. At the end of May 1943, the discharge over the cascades was 184,000 cu ft per sec—50% greater than at low water.

For Grass Island, numerical values of the discharge and diversion coefficients in Eq. 3 were obtained by using the Method of Least Squares, as previously explained. In order to complete the formula, it was necessaary to determine the value of the term K for the spring of 1942, before the causeway construction was begun or while it was in its early stages. For gages in the Chippawa-Grass Island pool, there is usually a period of about three weeks for determining the spring value of K, after the effects of the ice have ceased and before the start of the rise due to the seasonal cycle, but in 1942 the start of work on the causeway reduced the length of this period for most gages to about one week near the end of April. The complete formula for Grass Island is as follows:

$$G.H. = 560.13 + 0.217(\text{Morrison} - 336) - 0.310\,U.S. \quad (4)$$

In Eq. 4 the value of the Qn coefficient is taken as zero, for the reason that the Queenston diversion is by-passed around both the Grass Island and Morrison Street gages. It does not mean that the Queenston diversion has no effect on the water level at Grass Island but that a Queenston diversion of, say, 10,000 cu ft per sec would have the same effect as a reduction of 10,000 cu ft per sec in the discharge of the river. In formulas for gages farther upstream where the effect of this diversion is smaller, a term must be inserted for the Queenston diversion and the sign of its coefficient is positive.

The gage relation formula for Grass Island (Eq. 4) is used with the recorded Morrison Street gage heights, and the power diversion figures, to compute the mean daily gage heights that would have obtained at Grass Island if there had been no construction work and no seasonal cycle. The difference between these computed gage heights and those actually observed will represent the change due to the construction work, except that the effect of the seasonal cycle will be included.

The construction of the causeway was started in April 1942. By June 5, the causeway and the foundation of Tower Island were finished with a resulting rise of three-tenths of a foot in the water level at Grass Island. During December 1942, two 100-ft openings were made in the causeway at points where the water was deepest, and this lowered the water about one-tenth of a foot. Placing of rock for the weir itself started on September 8, 1942, and was continued until March 15, 1943, with a small amount of work remaining to be done during 1943. The net effect of causeway and weir at the end of May 1943 was a rise of seven-tenths of a foot at Grass Island with the comparatively high water then obtaining. Hydraulic calculations show that at standard low water the effect of the weir, without the causeway, should be close to the specified rise of 1 ft at Grass Island.

The rise of 1 ft, more or less, at the power intakes in the Chippawa-Grass Island pool will produce extra power because it will increase the carrying capacity of the power canals as well as give a small additional head on the plants. In addition, the rise in water level at the intakes helps to reduce winter ice troubles and consequent power interruptions. It has been estimated that the increase in firm power at all plants affected will exceed 30,000 hp. There are few other industrial areas where such a substantial amount of power could be secured with the use of such small amounts of critical materials.

Notwithstanding its present utilitarian function, the weir also will fulfill its original purpose, as envisaged by the Board, of helping to preserve the beauty of the Falls. The scenic value of the American Falls and the Three Sisters Islands will be enhanced by the resulting increase in the flow around both sides of Goat Island, and there should be some improvement in the appearance of the American end of the Horseshoe Falls.

Concrete Arch Roof for Kansas City Factory

Movable Forms Speed Construction of the Pratt and Whitney Engine Plant

By TRACY WOOD

SUPERINTENDENT, ALBERT KAHN, ASSOCIATED ARCHITECTS AND ENGINEERS, INC.

APPROPRIATELY enough, the Kansas City Speedway was the site chosen for that miracle of rapid construction, the Kansas City Plant of the Pratt and Whitney Aircraft Corporation. For this structure, one of the last industrial buildings designed under the direction of Albert Kahn, concrete arch roofs were chosen to save steel. This paper by Mr. Wood was originally presented before the Kansas City Section.

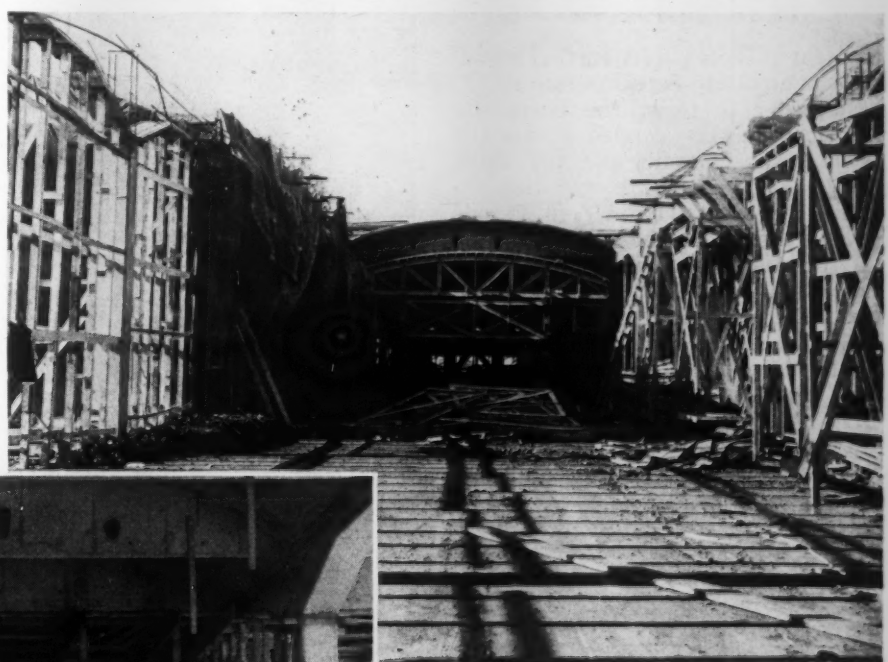
FOUR months from the time that ground was broken on a site at the Kansas City Speedway, thousands of men and women were at work in the mammoth Pratt and Whitney aircraft engine plant. Largely responsible for the remarkable progress schedule was the system of movable forms developed by the Long Construction Company and Turner Construction Company, general contracting organization.

In general there is nothing unique or remarkable about the construction of a concrete building. Except for the roof, this is true of the Kansas City plant. Basically it is a concrete slab floor laid on the ground, covered with millions of wood blocks to make a serviceable and efficient wearing surface. Over part of the area there is an extensive basement to house lockers and toilet rooms, cafeterias, and well-equipped kitchens. Around the building, brick curtain walls extend up to the window sills; above is wood sash, glazed with heat-resisting glass, a common type of construction. The roof itself will be especially described in this paper.

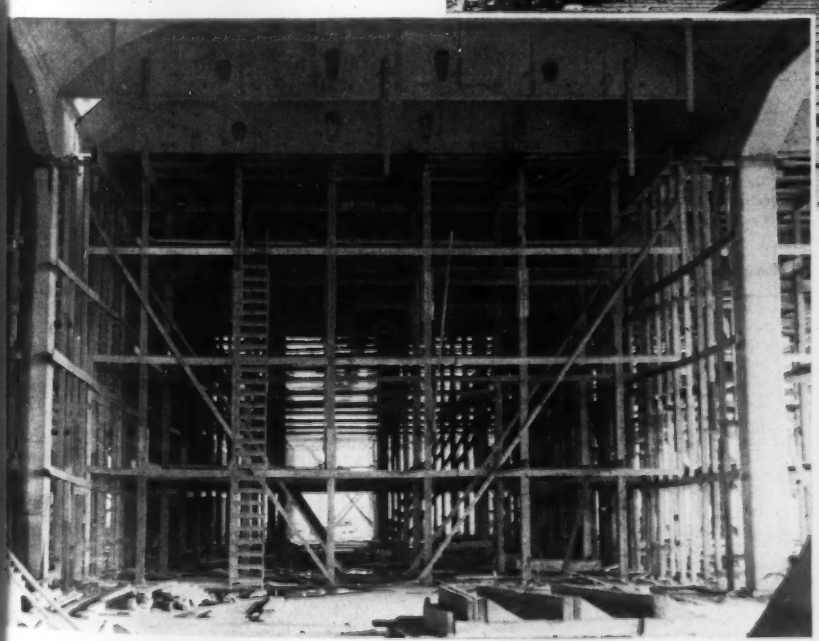
As an entity, the building forms part of the tremendous industrial plant of a nation at war. Our country needs planes for our combatant forces, engines are necessary for planes, and this building is one step in obtaining such engine production. In this instance, the problem was intensified by the shortness of time allowed and the restricted supply of material generally used in speedy construction.

Normally concrete is not considered as adaptable to rapid construction as structural steel. However, the design of this structure, a concrete arch roof supported on reinforced and continuous girders, was readily adaptable to the "assembly-line" method.

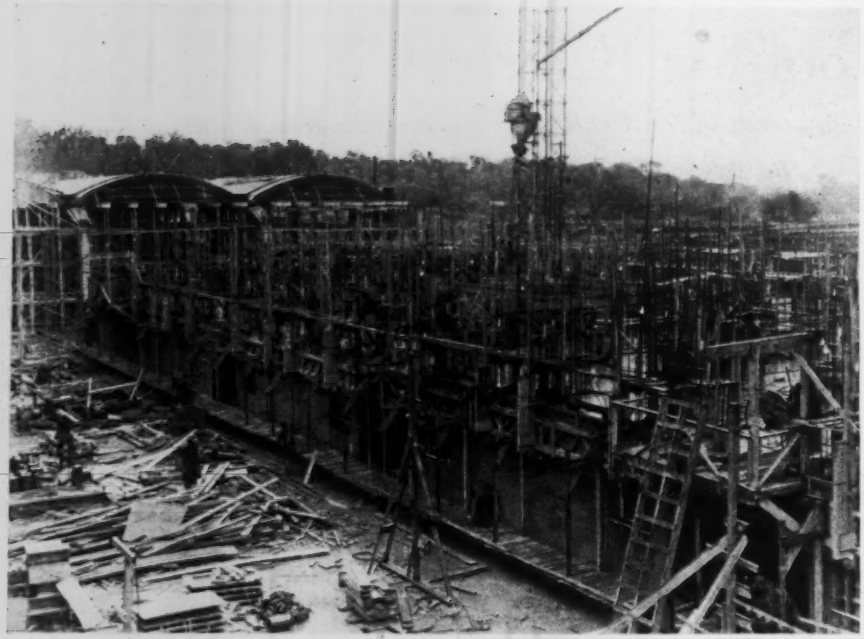
Although this design saves steel, a certain amount must be used, especially in the beam-and-slab sections of the building. In order to save steel, an arch roof was designed for the major part of the factory which puts the concrete primarily in compression. This permitted the use of a 3-in. slab, reinforced only with mesh, to span the 40 ft between girders. The girders were designed to take the vertical component of the arch reaction. These girders, were designed as continuous beams with haunches at the numerous columns and were reinforced for negative



(BELOW) FOR CONSTRUCTION OF SMALL END AREA OF ROOF, POSTED FORMS WERE USED Amount of Lumber Used and Time Consumed Were Much Greater Than for the Movable Sections



(ABOVE) ROOF ARCH FORM TELESCOPED AND READY TO BE MOVED TO NEXT SECTION Columns, Floor Slab, and Forms for Reinforced Girders in Place Ahead of the Roof Forms



SLIP FORMS IN PLACE FOR WALLS OF TEST CELLS
These Proved Both Speedier and Less Costly Than Fixed Forms

moment at these points, further reducing the amount of steel necessary.

In order to resist the horizontal thrust, ribs were carried across the arch at the columns and the third points. The structure was tied together by means of two $1^1/_4$-in. sq rods welded together to make a continuous tie from the face of the building to the expansion joint. In general, these ribs measured about 4 ft 8 in. from the under side of the slab at the crown of the arch to the bottom of the rib. This point is important as will be seen. With the exception of a few spaces on the roof for fan rooms and other purposes, this design is standard for the project and used a minimum of reinforcing steel.

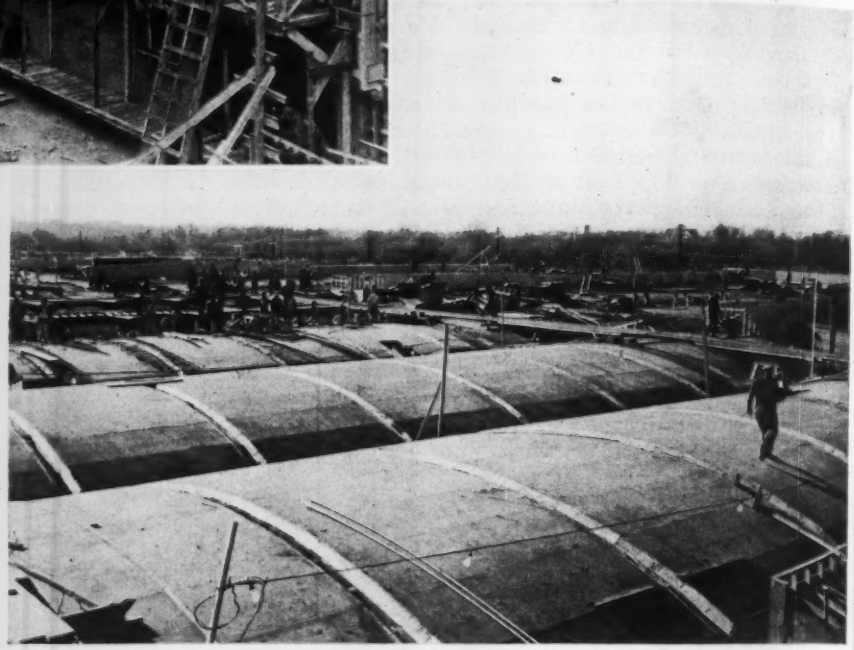
MOVABLE FORMS USED

One of the prime reasons for keeping to a standard design throughout the structure was the possibility of movable forms. These, of course, mean greater speed, and time was just as valuable as critical materials. Further, it is a critical component for which we have no substitute. The contractor is the one who uses the forms and is responsible for filling them with concrete. Therefore the job of designing these particular forms was given to the Long-Turner organization. The system proved to be simple and accurate for the placement of the nearly 50 acres of roof.

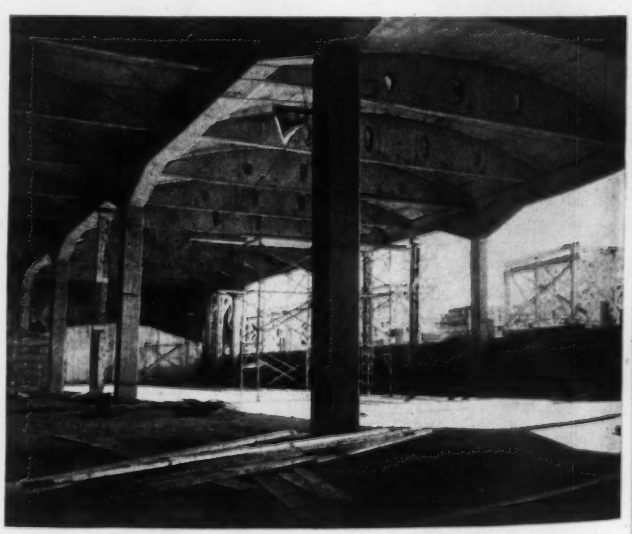
One form is 40 ft wide and 80 ft long and is in two parts. The lower part is built up of structural timber and is capable of moving along the building on wheels. This part might be called the carriage. The upper part can be telescoped down into the carriage about $5^1/_2$ ft. As has been mentioned, the ribs running across the arch are about 4 ft 8 in. deep, so this lowering is necessary to allow the top of the form to clear the bottom of the rib when the carriage moves forward. This lowering and raising of the upper part of the form is controlled by cables running over sheaves on the top of the carriage and fastened to chain jacks. There are eight jacks to a carriage and these are operated in unison. On the upper part of the form are wooden trusses, built to the curve of the arch. These carry the slab form, the forms for the tie ribs, and the sides for the girders.

CYCLE OF CONCRETING

To give an idea of the procedure, a complete cycle will be described. Out ahead of the forms, the columns have been poured and stripped, and the floor slab is in place. Forms for the bottoms of the girders have been placed, shored, and lined. The roof form from under the arch is now ready to be set and moved ahead.



TOP OF FORMS BEFORE CONCRETING
Wire Mesh Was the Only Reinforcing Used in the Arch Slab, Which Is Only 3 In. Thick

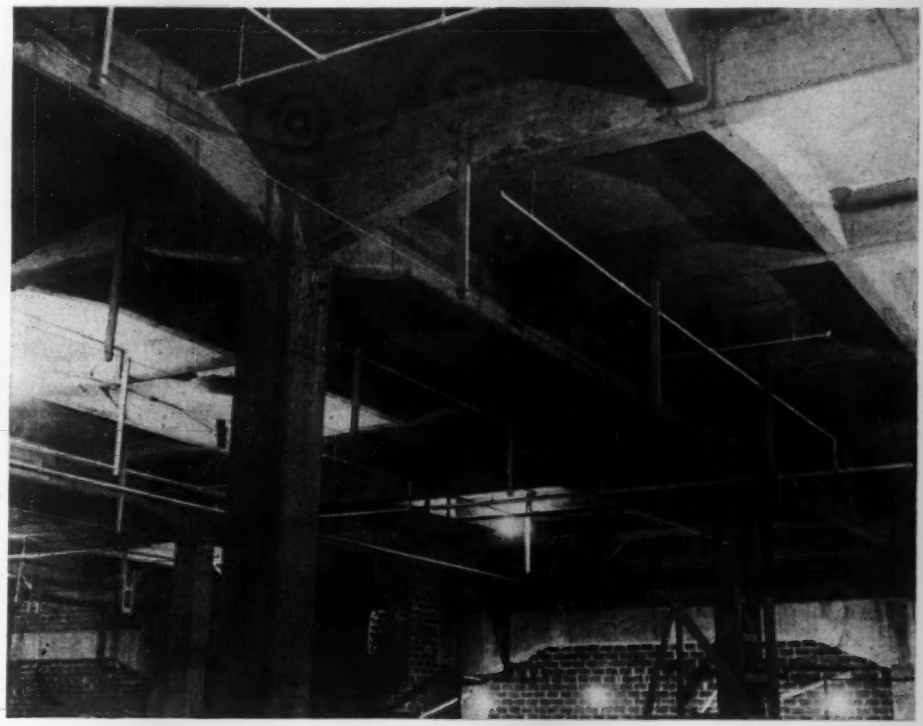


CONTINUOUS GIRDERS SUPPORT COMPLETED ARCH ROOF SECTION
Principal Function of Ribs Is to Take Horizontal Thrust; Holes in Ribs Provide for Piping

The sides of the girder are free, the rib sides and bottoms are loosened, and eight men are manning the cable jacks. The foreman blows a whistle and one point on the cable jack is released, inspection is made, nothing is found to be binding, and the whistle is blown again to indicate that another point is to be released. This continues until the upper part of the form is below the ribs. Out in front is a winch truck with its cable fastened to the lower part of the form. At a signal the whole form moves smoothly ahead 80 ft.

Again the jacks are manned and a blast of the whistle puts the form up in its new position at the required elevation. Safety blocks are installed (this form carries about 135 tons when poured), and men swarm aloft sweeping; repairing; setting girder sides, rib sides, and bottoms; placing inserts and reinforcing steel; welding the tie rods; and finally placing runways for the next pour. The concrete is hoisted and buggied into the forms.

Now comes a 48-hour wait until the cylinders which were taken from the pour are broken to be sure the concrete has enough strength to allow the form to be lowered. When this word comes, the cycle is again started. Under favorable climatic conditions for setting concrete, this cycle has been as short as five days.



SOME TWO-STORY SECTIONS WERE BEAM-AND-SLAB CONSTRUCTION
Post-Supported Forms Were Used for This Comparatively Small Area

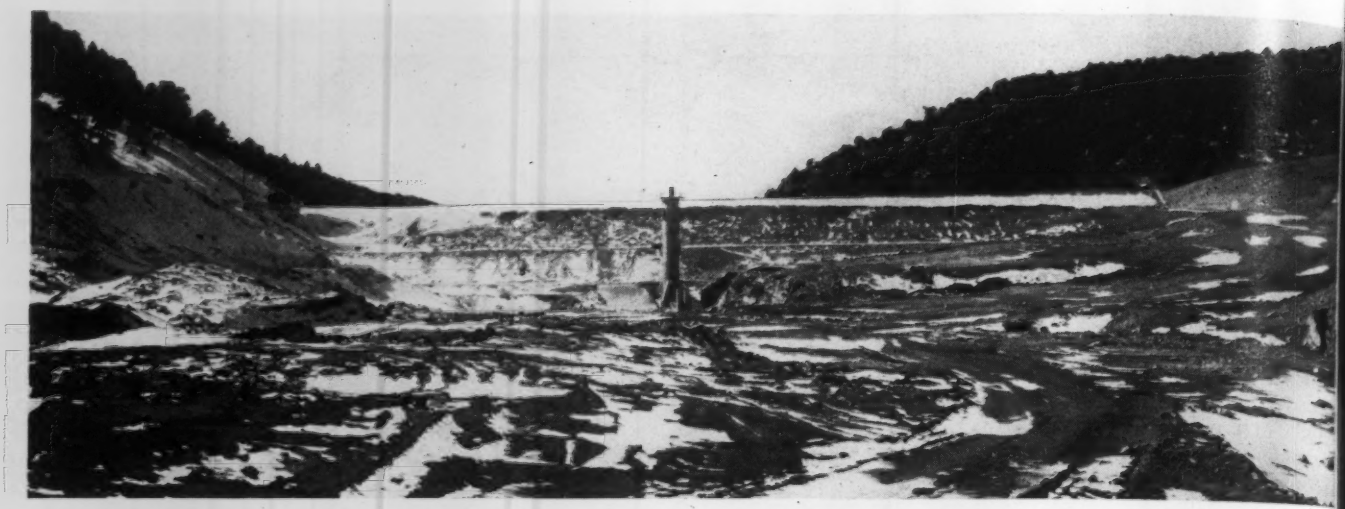
HORIZONTAL THRUST CUMULATIVE

Naturally, when the form is removed from the concrete, some deflection must be expected. This deflection results in an elongation of the tie steel resisting horizontal thrust. If allowed to accumulate across the entire width of the building, it would throw the outer columns out of plumb. To prevent this, about one bay in every four is held one cycle behind the adjacent bays, and this "orphan bay" is moved ahead only after the concrete on each side is supporting itself.

The forms extend the width of the building. Every five to seven days another 80 ft is added to that already placed. Even before the concreting was completed, machinery was installed and in operation. Much of the concrete was placed during freezing weather. Protection therefore presented another problem. A framework of tubular scaffolding was erected over the movable forms to carry tarpaulins. Salamanders furnished the heat required to prevent freezing. Concrete columns and outer wall surfaces were covered in the customary way with canvas until the danger of freezing had passed.

INTERIOR VIEW OF COMPLETED PLANT
Fluorescent Lights Installed and Wood-Block Floor Being Placed

Jobs like this are not done by any one man or one organization, but by close cooperation among many. C. L. Burns of Pratt and Whitney from the start guided the ideas into something tangible and worked with the Albert Kahn organization in the planning and designing. John Long, Assoc. M. Am. Soc. C.E., Bob Long, Herschel M. Parsons—all of the contractor's organization—were tireless in trying to get the job done quickly. On Bernard Browne of the Defense Plant Corporation fell the responsibility of the owner's interests.



NICHOLS DAM FROM THE RESERVOIR—INLET TOWER IN CENTER

Earth-Filled Dam at Santa Fe, New Mexico

Puddled Core of Nichols Dam Uses Material from Reservoir

By JOEL N. VAN SANT

ENGINEER OF DAMS AND RESERVOIRS, STATE OF NEW MEXICO, SANTA FE, N.MEX.

FOLLOWING six basic considerations of design for earth-fill dams, the structure here described was proportioned and constructed for stability. Although conditions at the site were generally favorable, no precaution was overlooked. Mr. Van Sant here gives the reasons for the design and construction adopted. His paper was originally presented at a recent meeting of the New Mexico Section in Santa Fe.

BUILT to create an additional water supply for the domestic and municipal use of Santa Fe, N.Mex., the Nichols Dam is a semi-hydraulic earth-fill structure, approximately 4 miles east of the city on Santa Fe Creek. The dam was designed and constructed during the fall of 1942 and early 1943 by the firm of Sanderson and Porter. The franchise for the water supply service is owned and operated by the New Mexico Power Company.

Prior to the construction of the dam, the plans for it and other necessary applications for water rights were submitted to the State Engineer's Office for his approval. Examination of the plans for the dam resulted practically in the redesign of the structure. There are certain essential features which must be considered before such a design can be accepted as satisfactory.

An earth dam is a structure composed of a mixture of silt, clay, loam, sand, and gravel or rock in suitable proportions to secure stability and watertightness. Several methods are used to obtain these results, but in the case of the Nichols Dam the semi-hydraulic method was adopted. In both the hydraulic and semi-hydraulic types, the placement of the fill is accomplished by means of water. While the control of the deposit varies somewhat in the two types, the essential difference is that with the hydraulic method the material is excavated, transported, and placed hydraulically, while in the semi-hydraulic method it is excavated mechanically and hauled to the site, where it is dumped on beaches. The material is then transported into the puddle core section hydraulically.

It is evident that when the material for the dam fill is at the same level or below that of the ultimate height of the structure, only the semi-hydraulic method can be used. Because at the site the two main deposits of material suitable for the fill were within the reservoir basin, it was necessary to use the semi-hydraulic method.

In his book, *Earth Dam Projects*, (John Wiley and Sons, Inc., New York, N.Y.) Joel D. Justin, M. Am. Soc. C.E., presents six features which should be considered in the design of earth dams: (1) spillway capacity, (2) line of saturation, (3) upstream and downstream slopes, (4) free passage of water through the dam, (5) a low velocity of seepage water, and (6) adequate freeboard. An attempt will be made to show how each of these criteria has been satisfied in the design of the Nichols Dam. The maximum height of the dam is 85 ft., the crest width 30 ft, and the maximum bottom width 440 ft. The maximum head of impounded water is 65 ft. The area of the reservoir surface is 32 acres, and the capacity 776 acre-ft. The dam embankment is composed of about 173,000 cu yd of earth and 4,000 cu yd of rock fill. The original estimated cost of the dam was $310,000, but the actual cost was below this by about $5,000.

First, the spillway capacity must be so great that there is no danger of overtopping. From a report on expected flood flows of the Santa Fe River, which was made for the National Park Service in 1939 by John H. Veale, Assoc. M. Am. Soc. C.E., then Assistant Engineer in the Service, the maximum discharge at this point was found to be 3,280 cu ft per sec. The designed spillway has a theoretical capacity of 3,600 cu ft per sec, which is believed to be adequate protection from overtopping.

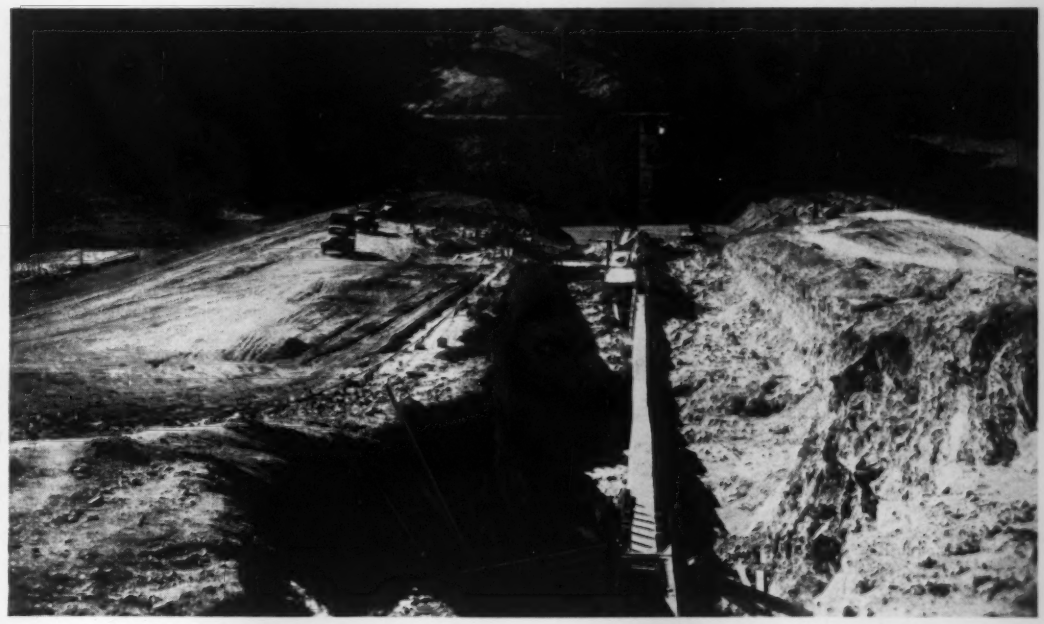
A rough check on the designed spillway capacity was made by using the formula, $Q = 3.087CbD^{3/2}$, which is for the solution of inlets and channels having vertical sides. Here Q is in cu ft per sec; C is a constant representing hydraulic efficiency, which varies from 1.0 to 0.82, accepted as 0.86 in this case; b, the width of the spillway bottom; and D, the depth of water over the spillway crest.

The cross section of the spillway inlet has a base width of 60 ft. On the right side of the entrance the re-

taining wall slopes on 1 horizontal to 5 vertical, which is duplicated on the dam side, but at this point there is a fin built in the retaining wall, which constricts the opening to a vertical section. Also it is proposed to add automatic drop-leaf gates in the inlet section later. For these reasons it was believed that the use of the vertical inlet formula with a hydraulic efficiency of 0.86 would give a reasonably conservative theoretical discharge value for the spillway.

The spillway is a channel excavated in rock so that the right side of the spillway retaining wall is natural rock or earth and the left wall is of concrete. Opposite the mouth of a small arroyo, which discharges into the spillway channel, the concrete retaining wall is quite massive and has an overhang section so designed that should the arroyo discharge with great force, the flow would be repelled and diverted back into the spillway channel. This channel is about 350 ft long. The bottom is of natural rock, and slopes on a grade of 8% from the spillway entrance for about half the distance down the channel, then changes to a 1% slope which continues to the end.

At the spillway entrance, the inlet is constructed of concrete wing walls with a concrete sill which has a footing in solid rock. The concrete sill, which forms the control section of the spillway, has been constructed so that flashboards can be placed in the channel as a temporary measure until the automatic drop-leaf gates can be acquired and installed. The concrete sill is 13 ft below the dam crest and 8 ft below the maximum water surface of the reservoir.



CUTOFF WALL EXTENDS TWO FEET INTO ROCK, FIVE FEET INTO PUDDLED CORE

SPILLWAY RETAINING WALL
Note Overhang Section to Deflect Water from Small Arroyo

LINE OF SATURATION

A second consideration was the line of saturation, which must be well within the downstream toe. This feature cannot be entirely satisfied because it is impossible to construct an earth-fill dam on an impervious rock ledge without having a slight amount of seepage at the toe of the embankment. For example, if a dam of fairly impervious material is placed on a highly impervious rock ledge, the water which tends to escape from the reservoir will seek the line of least resistance and will assume a hydraulic gradient in the dam so flat that it will intersect the downstream slope at a point considerably above the base of the toe. For all practical purposes, the hydraulic gradient and the line of saturation can be assumed to be one and the same thing.

In the design of the Nichols Dam, a puddle core of fine material was chosen for an impervious barrier. However, even though the best of materials is used and the dam constructed in the most exacting manner, a small amount of water will seep into the downstream portion of the structure. Hazard from this seepage was eliminated by placing a rock fill at the downstream toe. This fill not only allows the seepage to drain off rapidly without damage to the dam, but the extra mass and

weight aid in stabilizing the embankment, should the drainage prove ineffective and thus cause saturation.

A third condition is that the upstream and downstream slopes must be stable under all conditions. When wet, sand has an angle of repose of 2.5 to 1, and clay one of 3.5:1. A mixture of sand, clay, and gravel, however, has an angle of repose of 2:1. Therefore the dam of this mixture, which has been compacted and which has an average upstream slope of about 2.7:1 should be entirely safe. The hazard of wave action on the face of the dam, which can reduce gravel and sand to an angle of repose as flat as 7.5:1, has been counteracted by placing a 3-ft blanket of dumped quarry stone. This is sufficiently heavy and massive to resist the force of any expected wave action.

For the first 25 ft up vertically from the heel, the upstream face of the dam has a slope of 3:1. At this point there is a 10-ft berm. Up from the berm the face assumes a slope of 5:2 for the 35 ft to the top. A rock-fill embankment 15 ft in height has been placed at the heel for stability and drainage.

The downstream face slopes at 5:2 for about 20 ft up from the toe to an 8-ft berm. From this berm the slope is 9:4, which is interrupted by another 8-ft berm 35 ft higher; from this point to the top of the dam the slope is 2:1. There is also a rock fill at the toe of the dam 12.5 ft in height.

The function of the berm is to dissipate the precipitation which might accumulate on the face of the dam, thus preventing erosion, and also to spread the weight of the dam over a greater area of the foundation.

IMPERVIOUS PUDDLED CORE

There must be no opportunity for the free passage of water from the upstream to the downstream face. This fourth consideration is the reason for the main unit of the dam, the puddle core. This core has a maximum width at the base of 80 ft and tapers to a minimum of 20 ft at the top. The control of the puddle in this dam was excellent. The slopes of the core were kept steep enough so that the overlying relatively pervious material did not dilute it and the washing of the beaches was done with a minimum amount of water so that only the fines were washed into the pool. This is important because in a narrow pool, lenses of sand may be deposited in the core, so that they completely cross the impervious part of the dam and thus form channels through it. These lenses act as pipes and may, because of the increased velocity of the water passing through them, transport particles from within the dam and eventually cause the failure of the entire structure.

Even though the puddle core is constructed so that there is no danger of water flowing freely through it, there still remains a critical feature to be considered. This is where the puddle core ties into the foundation. The puddle core trench was excavated to firm stable schistose rock, which was cleaned by hand and left in the rough natural condition to afford a good bond between the foundation and the puddle core. Another natural condition, which aided the bond and will be a factor in keeping the dam from sliding downstream, is the fact that a major part of the foundation beneath the core slopes downward in an upstream direction. To tie the two different materials more securely together, a cutoff wall of reinforced concrete was constructed along the axis of the dam. This extends into the solid rock foundation a minimum of 2 ft and into the puddle core a minimum of 5 ft. In the excavation of this trench, care was taken to prevent shattering of the rock walls by using a minimum of powder where it was necessary, but the usual procedure was to bar and wedge out the rock by hand. No grouting was done because of the nature of the rock. This is fractured, it is true, but the seams are not interconnected for any great distance.

The axis of the dam was moved upstream from its original location so that less embankment would be re-

OUTLET CONDUIT FOLLOWS CONTOUR OF FOUNDATION ROCK
Collars Prevent Seepage Along the Conduit

quired, and also to decrease the cost of the right of way. This change of location proved of less advantage than had been anticipated, for upon excavation for the puddle core it was found that the foundation rock dipped sharply downward in an upstream direction so that considerably more excavation was required. This dip in the formation was caused by erosion previous to the deposition of the overburden, and also by the subsequent deterioration of the rock, which had reached a depth of as much as 20 ft in some places. It is probable that more thorough preliminary investigation would have been justified.

IMPERVIOUS MATERIAL BEING WASHED INTO CORE PUDDLE
Spillway Retaining Wall in Foreground

Another place in the dam where the free passage of water must be prevented is where the outlet conduit snakes through the embankment. This conduit is of reinforced concrete with an inside diameter of 5 ft. It was placed in the stream bottom and notched in solid rock, following along the toe of the right abutment and as a result has three slight curves in its length. The outer surface of smooth concrete pipe affords an ideal place for water to collect and flow along, so in addition to the cutoff wall, 8 diaphragms were placed about the conduit —two 4-ft diaphragms in the puddle core, one 15 ft upstream from the core wall and one 8 ft downstream from it; also three upstream in the relatively pervious part of the dam, and three downstream in that relatively pervious section. The only difference in the diaphragms in the pervious sections is that those downstream extend into the dam fill 4 ft and those above only 3 ft. These added collars increased the flow line of water along the conduit approximately 20%. This amount of increase has been found by experiment to retard the flow along the conduit so that it equals the velocity of the water flowing through the undisturbed dam fill.

VELOCITY OF SEEPAGE

The fifth consideration is that water passing through and under the dam must, when it rises to the surface, have a velocity so small that it is incapable of moving any of the material of which the dam or foundation is composed. As the Nichols Dam is constructed on a relatively impervious foundation, this consideration caused no worry. Even with the increased pressure from the reservoir on the underground water table, there would not be sufficient velocity to disturb this material.

To prevent the seepage water from accumulating at the surface of the bedrock on the downstream side of the structure and forming a weak place there, the path of the water was increased by the design of fairly flat slopes both on the upstream and downstream face. This path was also increased by the addition of berms on both faces, plus a generous top width. This increased water path will reduce the velocity of the water seeping through the embankment. Once the water does penetrate the impervious core and reaches the downstream portion of the dam, it is drained as quickly as possible through the porous material at the face of the dam.

This statement is not quite correct because the material on the outside face of the dam is somewhat denser than that just inside the surface. This is one of the bad features of a semi-hydraulic fill dam and cannot be helped because of the method used for transporting the material to the site. As stated before, the fill is dumped on the beaches from trucks and then washed into the puddle core by streams of water which are directed on to the dumped material.

This operation leaves a portion of relatively impervious material at the face edge of the beach, which is not desirable. Rock, as large as or larger than the size of a man's head, was sorted from the fill and placed at or near the face to increase the stability of the dam.

ADEQUATE FREEBOARD IMPORTANT

The sixth condition that must be filled is that the freeboard must be such that there is no danger of overtopping by wave action. This feature is a function of spillway capacity and is that vertical distance from the crest of the dam to the water surface of the reservoir when the spillway is discharging at its maximum capacity.

For larger reservoirs, Rufus H. Carter, Jr., Assoc. M. Am. Soc. C.E., who was Engineer of Dams before the writer, suggested the following formula, which he incorporated in the preliminary New Mexico State Dam Code:

$$H = 1.5[1.5(D)^{1/2} + 2.5 - (D)^{1/4}]$$

in which H is the freeboard in feet, and D, the length of the reservoir in miles. If one mile is used in the formula, the resulting freeboard is 4.5 ft; but in the rules and regulations of the State Engineer, the minimum requirement is given as 5 ft. Therefore a 5-ft freeboard was provided, because the reach of Nichols Reservoir is about three-quarters of a mile in length.

During the construction of the dam, a few changes in the plans were found necessary, but whenever such changes were made, it was to allow for conditions that could not be determined in the preliminary survey and investigation. These changes in no way are detrimental to the safety of the structure, and in most instances increased its strength and stability.

All requests and suggestions made by the State Engineer, Thomas M. McClure, M. Am. Soc. C.E., were given the kindest consideration by the engineer of Sanderson and Porter, Daniel Pelletier, and by Malcolm Heffelman, the engineer for the New Mexico Power Company. The photographs used with this article were taken by Harmon Parkhurst, of Santa Fe, N.Mex.

Huge Highway Project for Philadelphia

State and City Coordinated War and Postwar Planning

By JOHN U. SHROYER

SECRETARY OF HIGHWAYS, COMMONWEALTH OF PENNSYLVANIA, HARRISBURG, PA.

IN the largest highway program ever undertaken by the Commonwealth of Pennsylvania within a single city, postwar as well as wartime needs are anticipated. The design of access roads to navy yards, airports, and industries has been influenced by the need for expanded arteries to serve postwar traffic. Approximately $12,000,000 has been provided for the design and construction of this coordinated city and state program of highway improvement.

IMPRESSIVE as it seemed when originally projected, the huge state and city program for highway improvement within the municipal boundaries of Philadelphia, is today serving an even bigger purpose than its planners foresaw. By concentrating on those parts of the project which meet military, naval, and war-plant traffic needs, the program is serving the entire cause of the United Nations. Every possible effort and all available materials have been turned to immediate construction of wartime "access roads." The original single program has thus become in effect a two-part program, divided into (1) urgent wartime projects, and (2) postwar projects.

Anyone who has observed the inadequate intra-city highways leading, for example, to the Delaware River Bridge or the Philadelphia Municipal Airport, will understand the need for improvement. It was, in fact, largely to remedy such bottleneck conditions that the $12,000,000 State-City Program was designed, funds being provided by the state from the Motor License Fund.

Plans were prepared jointly by the Pennsylvania State Highway Department and the Philadelphia Bureau of Engineering. Construction work is supervised by the State Department of Highways, while the city acts as agent of the commonwealth in condemnation proceedings and other legal actions.

Three projects (Fig. 1) were marked for immediate construction, and three postponed until after the war. Obviously the three on which construction was begun were those that would most effectively serve the war effort. The first of these to be considered is the access road to the Philadelphia Municipal Airport.

ACCESS ROAD TO PHILADELPHIA AIRPORT

During recent years the city of Philadelphia has developed in its southwestern area one of the most modern airports in the East. At the beginning of the war, the airport was taken over by the Army and became an important part of our national defense system. Even before the war, however, the airport brought an immense increase in traffic to an area which previously had comparatively little traffic and, in consequence, boasted few modern road improvements. It soon became apparent that a thoroughfare capable of serving the airport adequately was needed. Studies of the situation showed conclusively that a modern roadway extending in a northerly direction was required. This would make possible quick access to south and central Philadelphia by way of Broad Street—a long, wide thoroughfare running the entire length of the city.

ACCESS ROAD TO NAVY YARD NEARING COMPLETION
Center Dividing Strip Has Reflecting Type Curbing

Thus on October 6, 1941, an agreement was made between the state and the city to open and construct Essington Avenue from Island Avenue, near the airport, to Passyunk Avenue and to continue the improvement along Passyunk Avenue to 63d Street. Here the project would join an existing improvement on Passyunk Avenue, which had an unlimited bridge over the Schuylkill River, and thus further provide quick access to South Broad Street and central Philadelphia. The length of this improvement is 1.96 miles, and the cost estimated at $1,000,000.

Since Essington Avenue was not open to traffic, no traffic figures were available to the engineers. However, since the improvement was designed to relieve the traffic situation on Penrose Avenue, it was decided to use the figures of a pre-war traffic count on that thoroughfare. These data indicated that on an average day Penrose Avenue carried approximately 27,000 vehicles, and on an average summer day, approximately 34,000.

To serve this traffic and expected increases, the cross section of Essington Avenue was designed to have a width of 108 ft and provide for four 12-ft traffic lanes, separated into two roadways by a 6-ft divisor. The cross section of improved Passyunk Avenue is 108 ft wide and provides four 12-ft-wide traffic lanes, separated into two roadways by a divisor of variable width. It is expected that the improvement will be completed and open to traffic during 1943.

In addition to opening an improved route from the airport to central Philadelphia, a need was also seen for an access road along Island Avenue in a northerly direction connecting to U.S. 13. This improvement, covering a distance of 1.23 miles, was also planned to provide access to local feeder routes in that area. Pre-war traffic counts indicated an average daily traffic on Island Avenue of approximately 14,000 vehicles, and a summer daily average of approximately 18,000 vehicles.

Here the following design was agreed upon. The cross section of Island Avenue—from Buist to Eastwick avenues—was designed to have a width of 118 ft, with provision for an unpaved street car area in the center, 28 ft wide and margined by raised curbing. On each side of this street car area provision was made for a 33-ft-wide roadway, comprising one parking lane 9 ft wide

and two 12-ft traffic lanes. From Eastwick to Tinicum avenues the cross section of Island Avenue was planned to have a width of 108 feet, providing for six 12-ft traffic lanes, separated into two 36-ft roadways by a 6-ft divisor. Provision was also made, of course, for necessary paving of street intersections.

The commonwealth will pay all damage and construction costs, which are estimated at $650,000, the city furnishing plans and facilities for determining damages. Construction from the airport to Eastwick Avenue was completed in July 1943. The remaining portion to Buist Avenue will require rearrangement of trolley line facilities, which has been approved by the Pennsylvania Public Utility Commission and for which plans are now being completed. It is expected that this part of the work will be finished before the end of the war, provided the use of large quantities of critical materials can be avoided.

Rapid development of the South Philadelphia area, which extends to the busy industrial city of Chester, has long indicated the necessity of an adequate thoroughfare to connect this area with West Philadelphia. Under existing conditions accurate estimate of traffic flow is not possible, but traffic engineers predict that an adequate thoroughfare will receive a daily flow of between 25,000 and 30,000 vehicles.

In order to provide these facilities, provision has been made for opening an extension of Oregon Avenue west from 18th Street to 23d Street, and opening Vare Avenue, from Oregon Avenue at 23d Street in a northwest direction to 34th Street, and on 34th Street north to Grays Ferry Road—a distance of 2.18 miles. The design provides for two cartways 36 ft wide and separated in the center of the street by a modern traffic divisor. The all-cost estimate is $1,000,000.

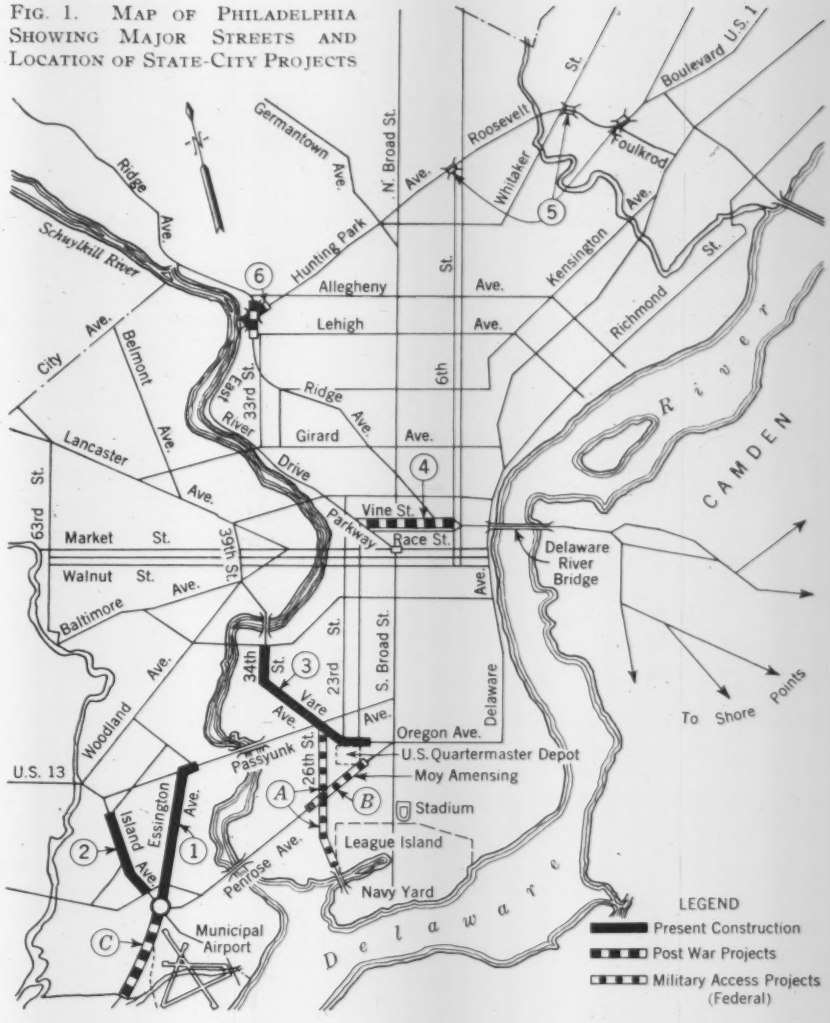
Work is now (August 1943) being completed by contract on the section of Oregon Avenue extending from 18th Street to 23d Street, and on Vare Avenue from 23d Street to Passyunk Avenue. Plans for the remainder of the project are also being completed, with federal approval strongly indicated.

In addition to the State-City Program, three separate road improvements are being constructed with state and federal funds. These projects, identified on Fig. 1 as *A*, *B*, and *C*, were authorized by the Federal Access Road Program as essential to war production.

Project *A* is designed for Navy Yard traffic relief, and costs approximately $784,000. The purpose of this project is to expedite the movement of employees between the League Island Navy Yard and Passyunk Avenue, a distance of 1.76 miles. The project also involves improvement of a portion of Penrose Avenue, where it is crossed by 26th Street for a distance of 0.4 of a mile. The Navy has constructed a lift-span bridge over the "Back Basin" and built all necessary road and parking facilities within the reservation.

To speed construction this project was divided into two sections. The first—from the Navy Yard to Penrose Avenue—was begun May 20, 1942, and is completed. It consists of two 24-ft lanes, divided. These are margined with earth shoulders and curbing on each side, and there is a sidewalk along the west side. A concrete and steel bridge, built to support three tracks of the Pennsylvania Railroad serving Girard Point, provides a highway underpass. Throughout the construction period, traffic to and from the Navy Yard was carried without interruption.

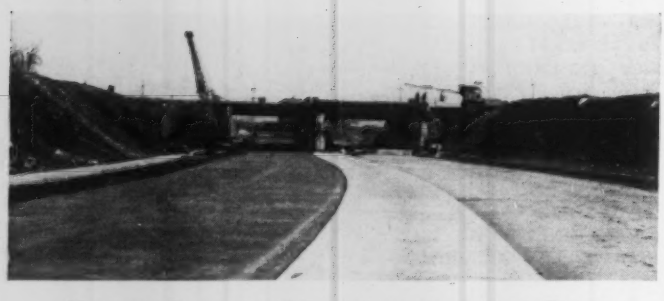
FIG. 1. MAP OF PHILADELPHIA SHOWING MAJOR STREETS AND LOCATION OF STATE-CITY PROJECTS



The second section—from Penrose to Passyunk avenues and including work on Penrose Avenue—was started March 22, 1942, and it is expected it will be completed by September 1, 1943. The 26th Street project consists of two 24-ft lanes, divided; the Penrose Avenue section, of two 36-ft lanes, divided. An additional 12-ft lane, 500 feet long, was added on the north side of Penrose Avenue to provide a stand-by lane for left-turn traffic to the Navy Yard.

Project *B* will also benefit the Navy Yard, by providing for the continuation of Penrose Avenue from the work already under way on the second section of 26th Street, to 19th Street and Moyamensing Avenue. Its purpose is to supply traffic facilities to South Broad Street and central Philadelphia. The improvement will be 0.75 mile long, and consist of two 36-ft sections, divided—with curbs, and sidewalks if the latter are considered necessary. This project is also being constructed as an emergency access facility.

Plans are now in preparation, but work cannot be started until the 26th Street projects are completed and carrying traffic through to Passyunk Avenue. It is estimated that four months will be required for construction, and the cost will be about $782,000.



HIGHWAY-RAILROAD SEPARATION ON ACCESS ROAD

Project *C*—a section of the Industrial Highway on Essington Avenue to join work already under construction by city-state agreement—is to be an access road from City Line at Bow Creek, to Island Avenue, at the Philadelphia Municipal Airport, a distance of 1.51 miles. The purpose of the project is to provide adequate facilities to war industries in both Philadelphia and Delaware County, to the Philadelphia airport, and to docks and shipping along the Delaware River. It will be constructed by the State Highway Department at an estimated cost of $868,000.

The improvement will be graded for a width of 108 ft, and two 24-ft divided lanes will be constructed at this time. A special feature of the work will be a traffic rotary at the intersection of Essington and Island avenues, with an interior radius of 150 ft and an outer radius of 204 ft. All plans have been prepared, and it is expected that construction will begin shortly.

A noteworthy feature of all these projects is the use of white cement reflector curbing along the traffic separating strips. The curb is normally 15 in. wide, with a smooth vertical face adjacent to the pavement 1 in. in height, and a scored face on a slope, with an over-all height of 6 in. When this curb design is used, the minimum width of the divisor in the center of the highway is 30 in. The divisor is formed by placing two sections of curb back to back. The reflecting curb has proved particularly effective in wet weather, because the film of water, acting like a mirror on the corrugated surfaces, intensifies the reflection. It is of great value also in fog where it not only supplies a line of light for the driver to follow, but keeps him from crossing the center of the road into the approaching line of traffic.

APPROACHES TO DELAWARE RIVER BRIDGE

Of the three postwar projects contemplated, the largest and most important is the approach to the Delaware River Bridge. This famous structure spans the Delaware River from Philadelphia to Camden, N.J. Immediately after the bridge was opened to traffic in 1926, an almost tragic inadequacy in the Philadelphia approaches became apparent, and in the ensuing years these approaches have become the worst bottleneck in the Philadelphia area.

Two one-way streets—Race and Vine—each with a cartway width of only 26 ft between curbs and with parking permitted on one side, provide the main east and west approaches. In 1941 these approaches were crowded with a daily average of 40,000 vehicles. On July 27, 1941, its busiest day, 83,014 vehicles crossed the bridge.

Plans to eliminate the bottleneck provide for the use of Vine Street from the bridgehead at 6th Street, west to the Benjamin Franklin Parkway at 20th Street, a distance of 1.25 miles. A loop on Ridge Avenue, south from Vine to Race streets, and east on Race Street, to the bridgehead was included. The estimated cost of all work was set at $7,500,000.

The Roosevelt Boulevard, extending east from North Broad Street, Philadelphia, to the Bucks County line, is a multiple-lane thoroughfare, 300 ft wide between house lines, with tree-shaded grass plots. Pre-war traffic on this famous highway varied from 20,000 to 25,000 vehicles per 24-hour day. This included traffic brought to the City by U.S. Routes 1 and 13, as well as local traffic.

The general cross section consists of a one-way lane on each outer edge for use of buses and commercial vehicles, and a wide center lane for two-way traffic. The latter is exclusively for fast-moving pleasure cars.

Three bridges over railroads were built at the time the parkway was constructed. Because these bridges accommodated only the center lanes of the boulevard, serious congestion and frequent accidents resulted, as traffic from the side lanes converged toward the center to cross them. To correct this situation, the structures are to be widened sufficiently to provide crossings for the outer lanes in a manner that will conform with the general plan of the boulevard. The right of way and plans for widening were provided by the city. The state agreed to bear construction costs, estimated at $850,000.

INTERSECTION OF HUNTING PARK AND RIDGE AVENUES

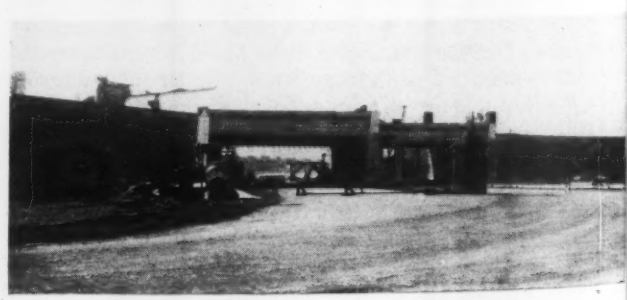
The intersection of Hunting Park and Ridge avenues, which leads from Philadelphia's famous East River Park Drive to both the Roosevelt Boulevard and northern sections of the city, constitutes a serious traffic hazard. Both streets involved are narrow, and they intersect on steep grades, with vision obscured. Apart from the high accident frequency, the resulting slow and difficult traffic flow causes annoying blockades during peak hours.

However, both streets are busy thoroughfares. In an average 24-hour day Hunting Park Avenue reached a pre-war volume of traffic in excess of 10,000 vehicles, and Ridge Avenue in excess of 8,000 vehicles. This despite the fact that many motorists drive extra distances to avoid the intersection.

To correct conditions, plans provide for Hunting Park Avenue to be placed under Ridge Avenue in a tunnel section, with retaining walls in the open-cut approaches. The cross section for the underpass provides two curbs, each 18 in. wide, and two traffic lanes 12 ft wide separated by a divisor 2 ft wide. The Hunting Park Avenue work will be approximately 2,000 ft long.

Ridge Avenue will be widened and reconstructed in this area for a distance of 1,900 feet. The new pavement will consist of two 24-ft sections, with a divisor 4 ft wide. There will be a new sidewalk 9 ft wide on the west side, and a new 11-ft sidewalk on the east side.

The necessary right of way has been acquired by the city, plans have been prepared, and the project is now ready for construction at an estimated cost of $600,000. The work will be undertaken after the war, when the various critical materials required for the work are again available.



REINFORCED CONCRETE BRIDGE CARRIES RAILROAD OVER 26TH STREET

The Design of Recent TVA Projects

VI. Cherokee and Douglas Projects

By J. C. NOWELL, ASSOC. M. AM. SOC. C.E.

PROJECT ENGINEER, TENNESSEE VALLEY AUTHORITY, KNOXVILLE, TENN.

SITES for these two power dams, Cherokee and Douglas, were so chosen that nearly identical structures were possible. This duplication, described by Mr. Nowell, saved many months in completing these projects which supply power for the manufacture of essential aluminum. The place of these two main tributary storage developments in the unified TVA plan was explained in the first article of this series, which appeared in the March issue.

BETWEEN the Cumberland and the Great Smoky mountain ranges is a wide hilly valley. Here the Cherokee and Douglas projects are located on the Holston and French Broad rivers, 53 and 30 miles respectively above their confluence, which forms the Tennessee River about five miles above Knoxville. Flood conditions and the available head, storage, and stream flow are identical at these two sites to such a degree as to permit practically complete duplication in design and layout. As a result major economies were secured in both time and expense for design, procurement of equipment, and construction.

The projects, which provide a combined storage of 3,200,000 acre-ft, to be utilized over a drop in head aggregating 500 ft in the main-river projects downstream, will add approximately 1.1 billion kwhr to the system output during an average flow year. While the immediate purpose of these projects is to increase the power output of the system during the emergency, they were designed as part of the integrated plan to improve navigation and prevent destructive floods, in addition to generating electrical energy.

At each of the dam sites the width of the valley formed by the river and its flood plain is approximately 1,100 ft. By the use of spillway gates, 32 ft deep, which had previously been developed for Watts Bar, ample space was available to locate the spillway and the power station with adjacent switchyard side by side in the valley floor. The conditions governing the layout were similar enough at both sites to permit arranging the structures in identical sequence. In this way the design of all junction blocks was also duplicated for the two projects.

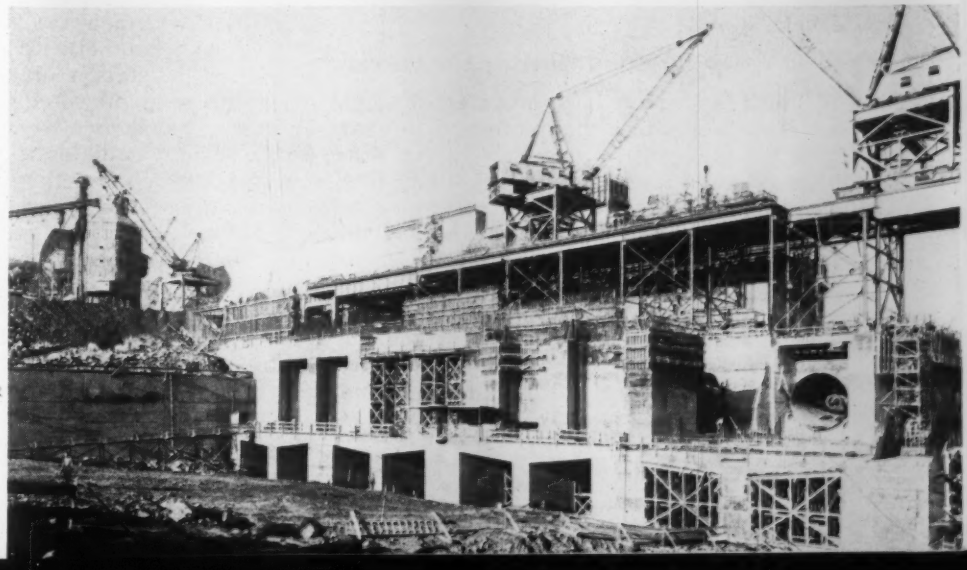
For both these projects the power station was located on the north bank of the river, with the switchyard immediately adjacent to reduce the length of generator and control cable runs. The longitudinal axis of the yard was placed parallel with the dam to obtain optimum conditions for the take-off of transmission lines and also for the handling of transformers, breakers, and other switchyard equipment. With such a layout, these can be rolled easily to within reach of the power-station gantry crane when repairs become necessary.

At each project provision was made for the ultimate installation of four generator units, three of which were authorized by Congress for the initial installation. However, because of the scarcity of critical materials, which developed due to war demands, one of the three units at each of these plants has been temporarily deferred. A stall for the possible future fourth unit was provided, with the substructure ready to receive embedded parts of the turbine, which will permit further increase in generating capacity if the streams are developed above these sites.

For these storage developments, straight gravity intake sections were designed to be stable independently of the power station concrete work and separated from the power station blocks by a contraction joint. An elastic filler was placed in this joint to avoid the possibility of throwing the units out of line from minor deflections of the intake blocks, as will result under varying reservoir loadings. A general cross section through these structures for the Cherokee project appears in Fig. 1. The same general arrangement was used for Douglas.

Penstocks were placed 61 ft on centers, as required by the power-station block spacing to provide proper embedment of the scroll cases. The use of corresponding block lengths for the massive gravity intake section would have encouraged the development of transverse cracks due to shrinkage effects. A layout was therefore developed, which would permit reducing the length of the intake blocks to $55\frac{1}{2}$ ft by varying the distances between the penstocks (61 ft on centers) and the center lines of the intake blocks in which they were embedded.

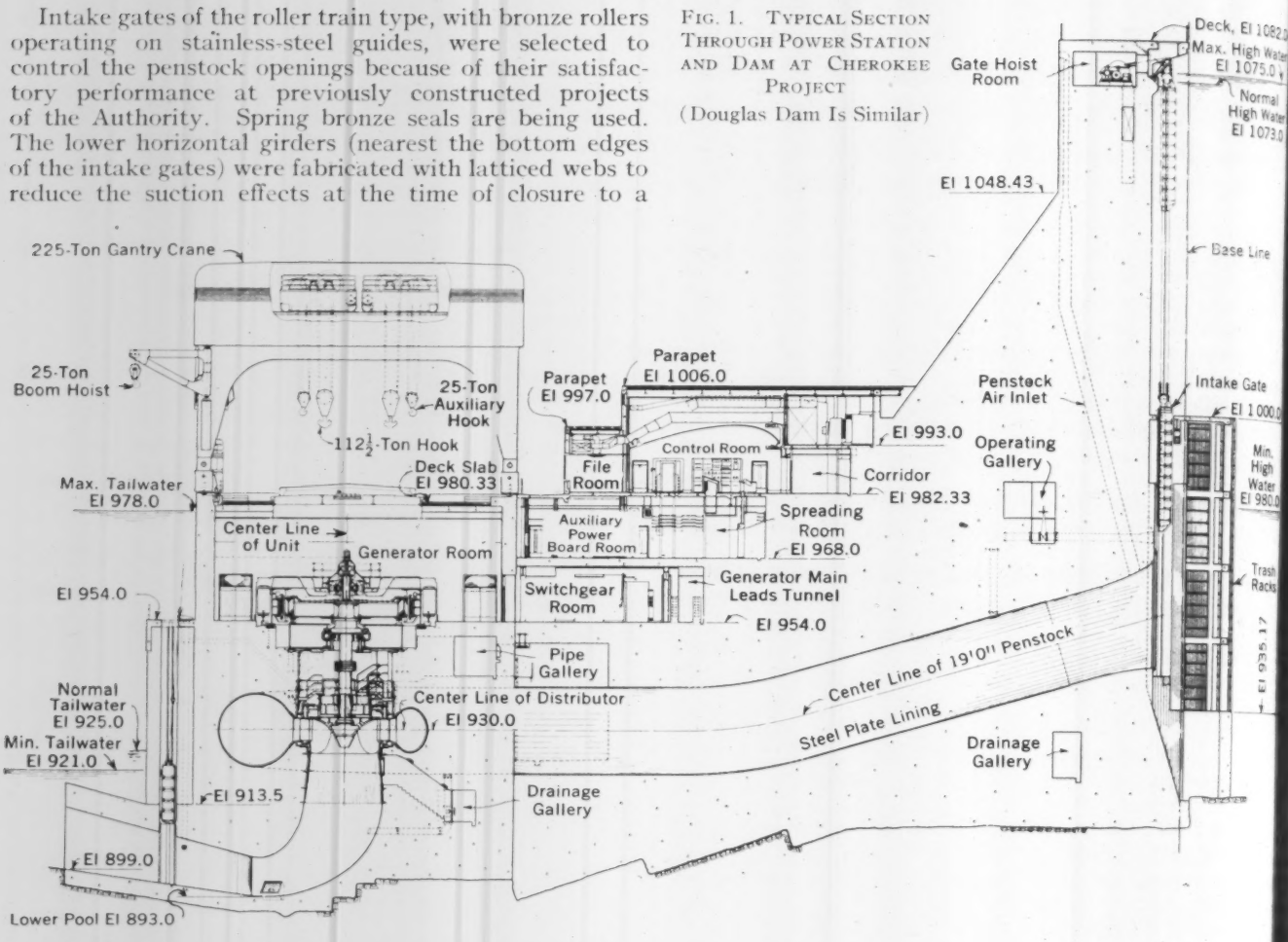
The openings to the penstocks, formed in the concrete in the massive upstream portions of the intake blocks, are transitions which vary from a rectangular opening to fit the intake gates to the 19-ft diameter of the steel penstocks used in the downstream section of the intake. Concrete support saddles adjacent to the contraction joint between the intake and the power station were designed to provide flexibility in the penstock lines for possible slight deformations of the intake sections. Final riveting of the connections between the penstocks and the plate-steel scroll cases was specified to be performed during relatively cold weather so that the temperature at the time of riveting will approximate that under normal operating conditions when the penstock is full of water.



DOWNSTREAM VIEW OF DOUGLAS POWER STATION DURING CONSTRUCTION
Penstock Provided for Future Unit Appears at Right

Intake gates of the roller train type, with bronze rollers operating on stainless-steel guides, were selected to control the penstock openings because of their satisfactory performance at previously constructed projects of the Authority. Spring bronze seals are being used. The lower horizontal girders (nearest the bottom edges of the intake gates) were fabricated with latticed webs to reduce the suction effects at the time of closure to a

FIG. 1. TYPICAL SECTION THROUGH POWER STATION AND DAM AT CHEROKEE PROJECT
(Douglas Dam Is Similar)



minimum, in cases where closure may be required while the turbine wicket gates are open. Hydraulic laboratory experiments, checked by field tests after construction was completed, have indicated that this open-girder design materially reduces closing suction effects, thus decreasing loads on the gate support chains and on the hoists, which are located in a gallery near the top of the intake section.

The decks of the semicircular trash-rack structures have been set several feet above the maximum expected reservoir drawdown level. Removable sections have been placed in the intake gate guides to permit swinging the gates clear from the guides and the face of the dam, and then lowering, if desired, to the rack decks for convenience in maintaining the gate. Such maintenance can usually be scheduled during periods of low reservoir level.

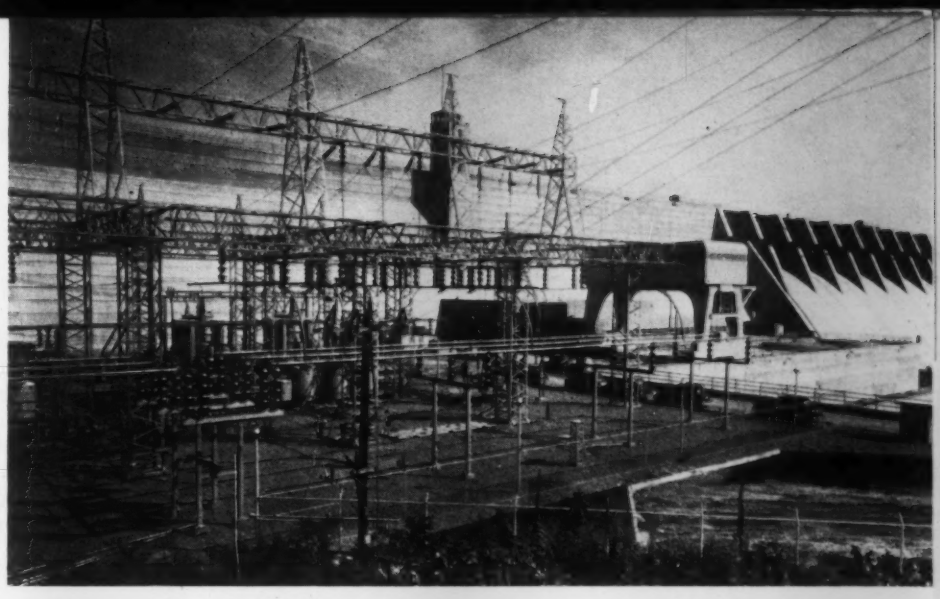
DESIGN OF POWER STATIONS

Trash racks have been provided of ample area to keep entrance losses to a minimum and to guard against any possible plugging by debris. On the other hand, since these storage reservoirs are relatively free from floating drift, it was not considered necessary to provide any special raking or other rack-cleaning facilities.

The general design of the power stations was determined principally by the equipment requirements, with special attention to facilitating construction because of the unusually fast schedules. Turbine and generator requirements set the generator-room floor level at approximately 24 ft below the maximum assumed tailwater. This relationship made it impossible to use the full outdoor-type power station; however, it favored a semi-outdoor type with a power-station crane of gantry design mounted on the roof.

Study showed that for the conditions applying, these semi-outdoor power stations would not only be more economical but would also have at least two other advantages in speeding up construction operations. First, the semi-outdoor type facilitated placing embedded parts for the turbines by means of the revolving cranes on the construction trestle, which was located over the toe of the intake blocks just upstream from the power station. The high superstructure walls, which would have been required for a fully enclosed type of power station, would have interfered with such early placing of the embedded parts. Second, use of the semi-outdoor design made it easy to provide more than the conventional amount of space for the assembly of the generator rotors and other power-station equipment by the regular power-station crane. The crane rails were extended on the bank some 40 ft beyond the regular erection space at the service bay. This additional 40 ft of space thus made available to the crane has proved particularly valuable in speeding up the assembly of heavy power-station equipment.

Heads available at these two storage sites vary from about 55 ft at the lowest anticipated drawdown to about 150 ft at the maximum, with normal heads of about 100 ft. As best meeting these conditions, 33,333-kva generators were selected, direct-connected to Francis-type vertical-shaft turbines having a normal speed of 94.7 rpm and rated at 41,500 hp at a 100-ft head. These turbines will actually develop the rated generator capacity with a head



LOCATION OF SWITCHYARD ADJACENT TO POWER HOUSE SHORTENS POWER CABLE RUNS AT CHEROKEE DAM

of about 97 ft, slightly less than the average expected. The center lines of distributors are set 6 ft above the minimum tailwater, which reduces the required excavation for the draft tubes and tailraces without inviting runner cavitation.

Because of the arrangement of the power station and intake structures, it was both economical and convenient to locate the electrical bay over the toe of the intake blocks just upstream from the power station. This bay contains electrical equipment such as main low-voltage switchgear and connections, generator neutral equipment, excitation cubicles, and auxiliary power transformers and switchboards. This location of the equipment permitted short generator leads and switchyard connections. Also, the control room was then centrally located on the deck over the electrical bay, and at the end nearest the switchyard.

The 225-ton gantry crane, which is mounted on the roof deck of the power station, serves to handle all major pieces of equipment as well as to operate the draft-tube gates. A transfer track for handling the main transformers extends from the service-bay roof deck out through the switchyard, the transformers being rolled from their foundations onto a car and transported to within reach of the gantry crane, which can then lower these large pieces into the service bay for untanking and repair.

CARE TAKEN IN LOCATION OF SPILLWAY

For these main tributary storage developments, required flood discharge capacity has been determined at or in excess of 300,000 cu ft per sec after careful flood studies, including the routing across the drainage areas of storms of maximum expected severity. Because of these high discharge requirements, particular attention was given to locating the spillway section.

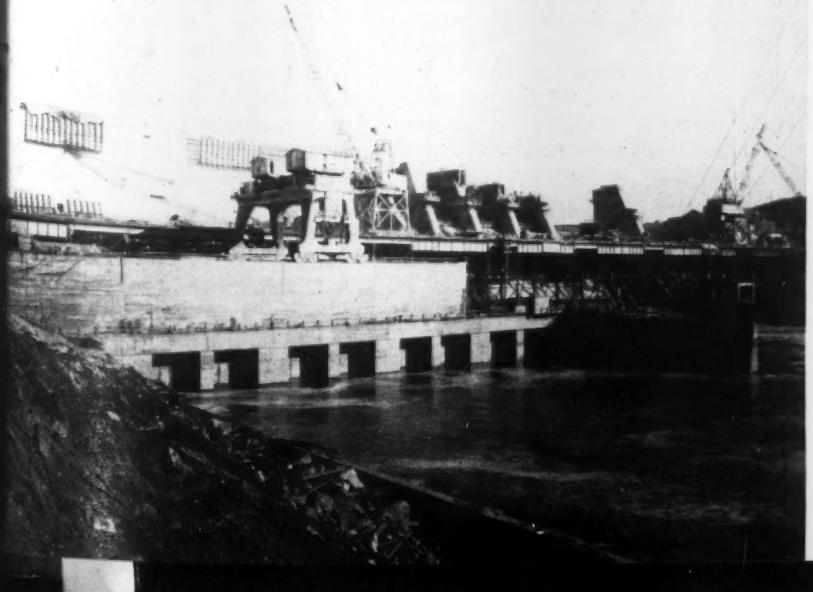
Principally to facilitate the required drawdown of the storage reservoirs before expected floods, a number of sluices are provided at the base of the spillway sections. These also supplement the spillway crest in meeting requirements for the passage of the maximum anticipated flood and can be operated to release water to assist navigation. Slide gates, 5 ft 8 in. by 10 ft, placed in tandem, were provided for the sluices. The upstream gate is used only for emergency closure in case of difficulty with, or required maintenance on, the service gate downstream. While these sluices will operate relatively infrequently, it was decided that their design should be carefully investigated, including hydraulic model tests, to eliminate possible cavitation difficulties. As a result of these tests it was found that a 15% constriction at the sluice exits would insure positive pressures in the conduits for all operating conditions.

For crest control, radial gates 32 ft high by 40 ft long and with 6-ft 6-in. intermediate concrete piers, are provided. As a result the spillway blocks are 46 ft 6 in. long. These gates and piers, also the I-beam lock-type deck above, the traveling hoists for gate control, and the outline of the spillway crest were all made similar to those previously developed by the Authority for use at Watts Bar and Fort Loudoun.

Because of the relatively high head and large required flood discharges, extensive hydraulic laboratory model tests were necessary in developing satisfactory designs for spillway aprons, particularly where sound rock was found to lie close to tailwater elevation, thus making it economical to place the apron at a relatively high level. Considerable difficulty was encountered in handling the high-velocity jets from the sluices, as the model tests showed that these would tend to sweep tailwater out of the apron pool with considerable violence, causing erosion of the rock downstream from the apron. Good conditions were finally secured by a combination of measures which included the following:

1. The sluice exit nozzles were simultaneously flared horizontally and flattened vertically to spread the jet over the maximum width of the apron.
2. The apron itself was lowered at the bucket to create a deeper pool for dissipating the energy of the jet.
3. The sluice discharge nozzles were bent downward so that the jet would be absorbed by the bucket pool.
4. A number of steps were provided in the apron, where it rises from the bucket to normal grades, further to diffuse jet energy.
5. A relatively high weir was provided at the downstream end of the main apron further to break up the flow and also to increase the depth of the pool.
6. Concrete paving was placed immediately downstream from the weirs.

Other design complications were involved at Cherokee, where it was necessary to omit sluices from the end blocks of the spillway section to leave these clear for rapid concrete placement in making closure after these blocks had served as river diversion slots. For this reason it was



COMPLETED POWER STATION AT DOUGLAS DAM

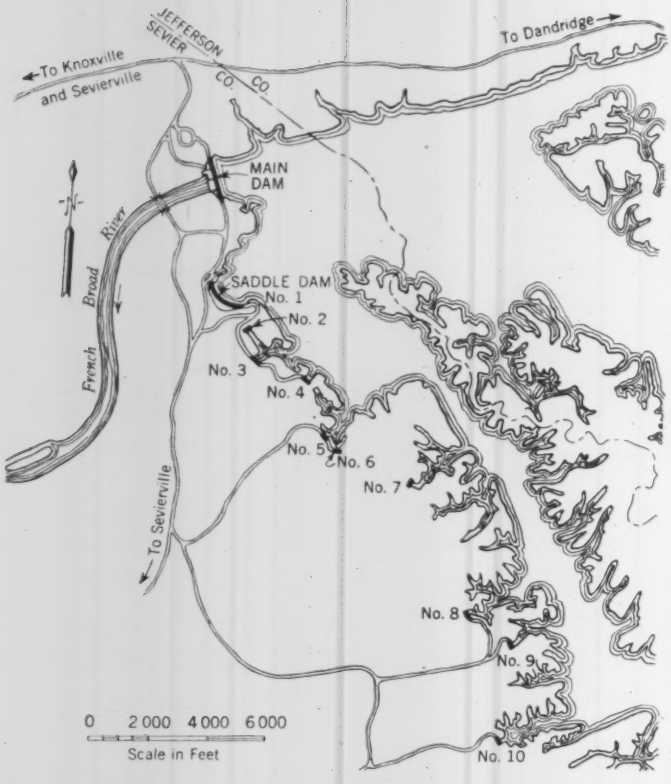


FIG. 2. SITE PLAN OF DOUGLAS PROJECT, WHERE ELEVEN SEPARATE DAMS WERE NECESSARY

necessary to install two sluices in a single block at each end of the sluice section. To reduce eddies in the apron pool, the exits of the two end sluices were angled out slightly, thus giving flow conditions approximating those that would have been secured with a conventional layout of one sluice per block.

EMBANKMENT MATERIALS AT THE TWO PROJECTS

At both the Cherokee and Douglas projects, soil investigations disclosed ample available borrow-pit materials in the range of fine silty clays desired for the central watertight portions of the embankments. On the other hand, in spite of a very thorough search, no adequate deposits could be found with a grading sufficiently coarse to permit conventional slopes for the outer shells of the embankments. It therefore became necessary either to use relatively flat slopes or to adopt special embankment designs. At Cherokee, after considerable study of possible embankment sections, the choice of designs lay between an all-earth section using an upstream slope of 1 on 5 and a drained downstream slope of 1 on 3, or an alternative section with upstream and downstream slopes of 1 on 2, using quarried rock shells to protect and support an impervious rolled fill. Because the main embankments at Cherokee were constructed on top of a shale ridge, the flat slopes would have greatly extended the distance out to the embankment toes and increased the quantity of earth fill. Under such conditions, a section using a central rolled-fill core with heavy rock-fill shells proved more economical and was adopted.

At the Douglas project (Fig. 2), no embankments were required at the ends of the dam, but numerous saddle dams were necessary to plug low gaps in the reservoir rim. The main dam in this group is Saddle Dam No. 1, which lies in a relatively flat draw in the rim. An embankment 87 ft high was required, and an all-earth section was found most economical. A 1 on $4\frac{1}{2}$ upstream slope was used, while a pervious drainage blanket was provided under the downstream portion to lower the saturation line, thus permitting use of a 1 on 3 downstream slope.

Tests proved that the natural overburden at the embankment sites of both these projects would not provide suitable foundations. Fortunately, this overburden extended only to relatively shallow depths, except under the downstream shoulder of the south embankment at Cherokee, next to the non-overflow dam. The overburden was therefore easily removed, stripping being carried to reasonably firm underlying shale or rock to provide foundations of unquestionable suitability. At the deep section of the Cherokee south embankment, the overburden was sloped partially back under the rock shell to provide a shoulder for the downstream portion, thus reducing excavation as well as rock material required for the shell and at the same time providing adequate safety.

RESERVOIR ADJUSTMENTS REQUIRED

Highway relocations in the reservoirs involved several main federal highway routes, particularly in the Cherokee Reservoir, where many miles had to be readjusted. In the Douglas Reservoir, highway relocations were not as extensive but several large bridges required raising or replacement.

The most important road relocation in the Cherokee area consisted of approximately 29 miles of state and federal highways involving U.S. Highways 25E and 11W. Two important new bridge structures were required on these routes—the German Creek Bridge with an overall length of 986 ft, and the Holston River Bridge, 1,674 ft long. In addition, numerous smaller drainage structures and approximately 68 miles of secondary roads were reconstructed above reservoir levels.

In Douglas Reservoir only about 4.5 miles of federal and state highways were affected, principally on U.S. Highway 25E. On this route the Walters Bridge, 1,767 ft in length, had to be raised approximately 10 ft above its original level. Another major readjustment was the Swann Bridge, 2,562 ft long, on U.S. Highway 70, which was raised approximately 40 ft. It was also necessary to provide a major new bridge for the reservoir crossing of the Southern Railway and to raise and relocate approach trackage. Among the many structures required on the 61 miles of county and access roads relocated around this reservoir, was a new bridge near Dandridge, Tenn., with an overall length of 1,506 ft, replacing the previous low-level structure which is being salvaged and reerected just below Douglas Dam.

A large rock and earth-fill dike, about 75 ft high at the deepest section, was required to protect the town of Dandridge, a large part of which would have been inundated without this special protection. Two principal creeks, with a total watershed of about 550 acres, joined near the center of the business district and flowed through a narrow defile into the French Broad River. A small diversion dam on each creek was constructed to intercept the major portion of the drainage from the watershed enclosed by the dike, and this flow is carried in long conduits around the ends of the dike. A small area consisting of about 55 acres immediately above the dike, not covered by the diversion conduits, required the construction of a pumping station. Adjustments were necessary also to the sanitary sewage system and streets in the city. Provision was made for possible future developments in the vicinity of the dike, as it is expected that Dandridge will become a center for boating and similar recreational use of Douglas Reservoir.

Above and Under Ground

Incidents from a Long Engineering Experience Around New York

By JOHN H. MYERS, M. AM. SOC. C.E.

LOOKING back on a long career of public service in the metropolitan area, on water supply and particularly on subway work, in which he was engaged for many years as Division Engineer, Mr. Myers has jotted down some of his experiences. This personal recording which, by the way, many engineers might emulate with profit, was meant particularly for the benefit of his children. But it is believed to be of interest to others, even those to whom he and his work are unknown. The few extracts given here are typical of this interesting engineering biography.

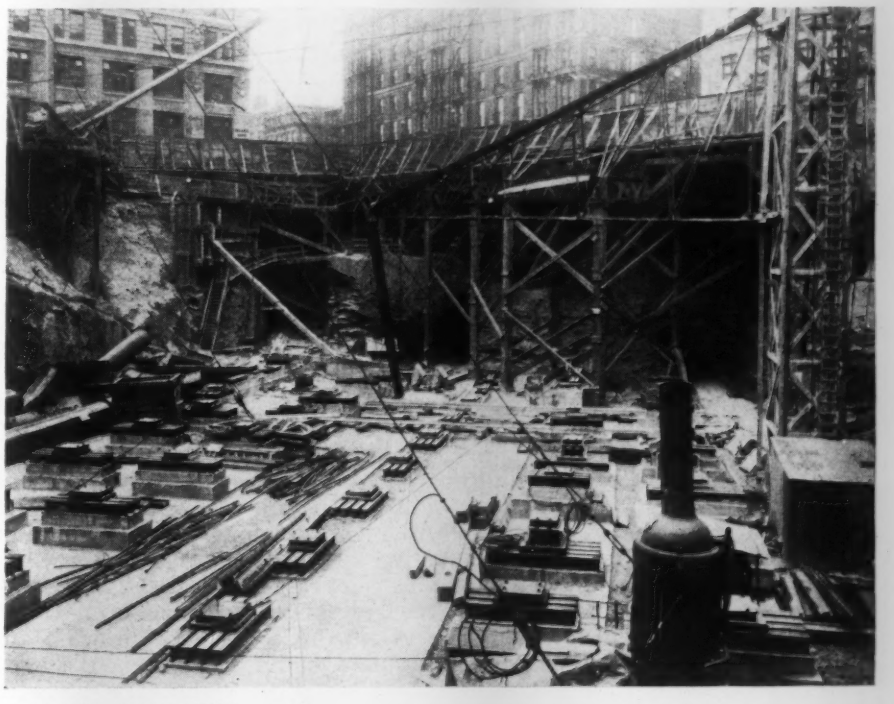
IN September 1894, having finished a stream gaging assignment on the Hudson River near Glenn Falls, N.Y., I was out of a job. Luckily this state of affairs did not last long. After a series of interviews at the offices of the Commissioner of City Works, in Brooklyn, I was referred to Arthur S. Tuttle (later President and Honorary Member of the Society, then principal assistant to I. M. de Varona, Engineer of Water Supply). Thus began a connection with the City of New York that was to last nearly 40 years. About 6 years of this was spent on the Water Works of the City, later the Borough, of Brooklyn. Another assistant engineer was W. W. Brush. Both Messrs. de Varona and Brush later became chief engineers of the Department of Water Supply of the City of New York, while Mr. Tuttle was to hold the responsible post of chief engineer of its Board of Estimate. All were members of the Society, of course.

On this work I had one of those infrequent opportunities to resurrect and use the principles of descriptive geometry, taught to all engineering students. It had been decided to increase the carrying capacity of the Brooklyn water conduit by laying a double pipe line parallel to it; from the Spring Creek Gate House to the Ridgewood Pumping Station, two 4-ft cast-iron pipes were laid. Charles D. Ward, M. Am. Soc. C.E., who at the time was one of the engineers of the Brooklyn Water Works, delighted in handing out problems to a cub engineer like me, and he asked me to work up the data for cutting the 4-ft cast-iron pipes, where they entered the conduit, so that their ends would fit the inside of the conduit or aqueduct. I fell back on my descriptive geometry, worked up the data, submitted it, and promptly forgot it.

A while later Mr. Ward took a short vacation and I was directed to take over his work during his absence. Before leaving, he showed me where he had recorded in one of his field books the data for cutting the pipe, and told me to use two cracked pipes, because this would eliminate an embarrassing question which was pending as to who was responsible for the cracking, the city or the contractor. The contractor's superintendent was not keen to use these pipes, as it involved carting them some distance. In my mind I can still see the contractor, John McNamee, an honest and very able man, standing as solemn as a judge, the superintendent on one side of him and I on the other, listening to our arguments pro and con as to the use of the cracked pipes. After we had our say, Mr. McNamee said tersely, "Jim, bring up the pipes." This of course the "super" did, and I then proceeded to lay out the cuts on them.

PROOF OF THE PUDDING

The pipemen who had been detailed to do the cutting were standing by with their dog chisels and sledges, and when I had completed the marking, they gave me a look which said louder than words, "you —— fool." It was, I will admit, a crazy-looking cut, but there was nothing to do but proceed, and the pipes were duly cut. A rope "sling" was put around one of them, and this the derrick picked up and lowered into the deep trench. As I looked over the side of the trench, my courage oozed away, because it seemed to fit not at all. But a bright thought struck me and I directed the foreman to adjust the sling to one side and to hoist easily so as to roll the pipe in the trench. This



Board of Transportation, N.Y.C.

DIAGONAL STATION CONSTRUCTION ON SITE OF GRAND UNION HOTEL, 42d STREET AND PARK AVENUE, LOOKING SOUTHWEST

Grillages to Carry Heavy Building Loads. Three Tunnel Excavations in Background: Left, Northbound Local; Center, Two Express Tracks; Right, Southbound Local. Latter Tunnel Caused Misgivings as to Alinement, Especially Adjacent to Buildings in Background—Belmont and (Center) Murray Hill Hotels

brought the pipe right side up and the fit was perfect. I was a much relieved young man, because I had begun to believe that I had been guilty of some grave error.

To complete my story, let me say that to permit the removal of the wall of the conduit where the pipes entered, a cofferdam had been built in it, which prevented any outflow of water. In this cofferdam, a bricklayer began the work of "making up" the masonry around the ends of my beautifully cut pipes and in due course had bricked himself in. It had been planned to get him out over the edge of the cofferdam by means of a rowboat, which had been left for the purpose upstream in the conduit at the Spring Creek Gate House. But those who went to get it found it not, and it was finally discovered that it had been removed to the Ridgewood Reservoir. So it was necessary to take it out of the reservoir, cart it back to Spring Creek, launch it in the conduit, and row it to the Ridgewood Pumping Station before the bricklayer could be retrieved. He was ultimately rescued after being confined several hours in a 2 or 3-ft space between the conduit wall and the cofferdam, past which was flowing the total water supply of the City of Brooklyn.

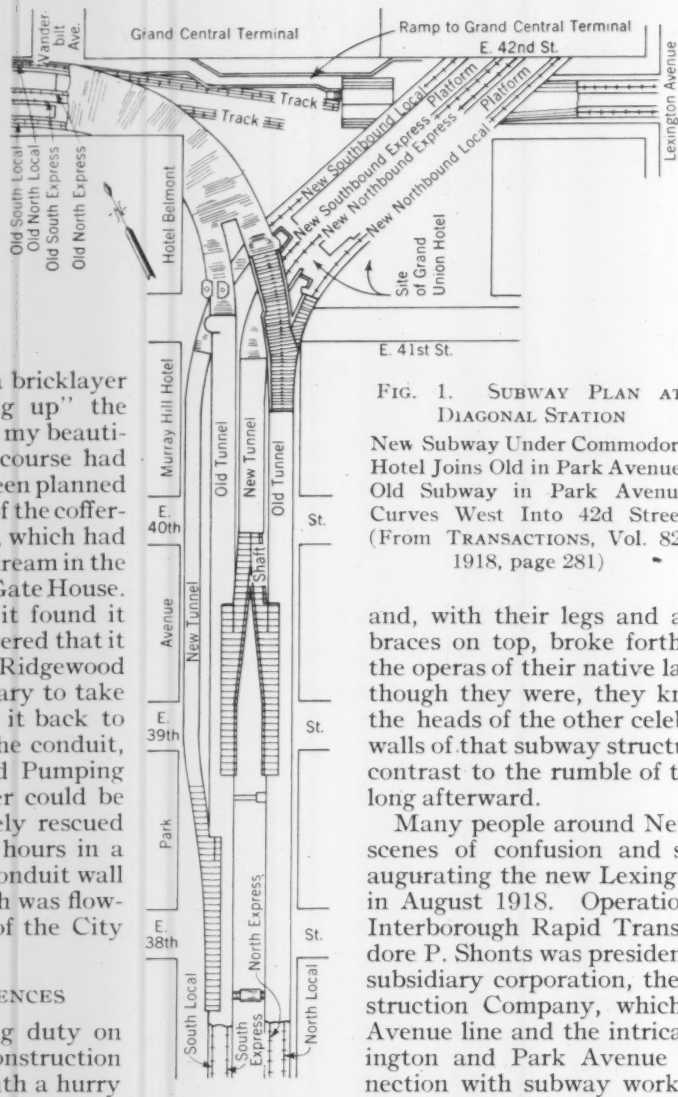


FIG. 1. SUBWAY PLAN AT DIAGONAL STATION

New Subway Under Commodore Hotel Joins Old in Park Avenue; Old Subway in Park Avenue Curves West Into 42d Street (From TRANSACTIONS, Vol. 82, 1918, page 281)

EARLY SUBWAY EXPERIENCES

I reported for engineering duty on the rapid transit subway construction on May 1, 1900. That month a hurry call came to "cross section" the surface of Broadway in the vicinity of 157th Street. I mention this incident because of the positions of responsibility ultimately attained by the men who did that prosaic work. Each was then, or became later, a member of the Society; one was chosen President and one a Vice-President—not a bad showing for an inconspicuous engineering party. Led by Alfred Craven, then Division Engineer, Messrs. Robert Ridgway, R. H. Jacobs, and I did this surveying before the contractor's plow destroyed the surface. Mr. Craven "ran" the level; Mr. Ridgway used the rod and recorded the figures; while Jacobs and I handled the tape. Mr. Craven ultimately became chief engineer of the subway work. Mr. Ridgway became a division engineer and as such supervised the building of the first subway in Brooklyn and the tunnel from Brooklyn to the Battery; later he became chief engineer. Mr. Jacobs, when he became head of the Track Division on the subway work, planned and supervised the laying of the greater part of the tracks in the subways and on the city-owned elevated railroads.

Celebrations were often held—sometimes quite formal and sometimes utterly spontaneous—to commemorate completion of some part of the subway work. One event at which I was present was unique. Although a few officials were there, the great bulk of those in attendance were laborers, foremen, superintendents, and engineers who were working on the job. The contractor set up long tables in one of the trackways of the Lexington Avenue line and loaded these with food and potables such as young and physically active men can assimilate.

No officials, silver spades, gold spikes, silver-tongued orators, or choruses of school children were there—just plain workmen celebrating the results of their intelligent toil. High and low fraternized and, with much hand shaking and back slapping, thoroughly enjoyed the occasion. Many of the Italian workmen, spurred no doubt by some of the more powerful potables, "shinned" up the subway columns and, with their legs and arms twined through the knee braces on top, broke forth in song, with selections from the operas of their native land. Hard-handed sons of toil though they were, they knew the Italian opera. Above the heads of the other celebrants they made the roof and walls of that subway structure echo with sounds in strange contrast to the rumble of the trains to be heard in it not long afterward.

Many people around New York vividly remember the scenes of confusion and subway jams incident to inaugurating the new Lexington and Seventh Avenue lines in August 1918. Operation of the lines was under the Interborough Rapid Transit Company, of which Theodore P. Shonts was president. He was also an officer of a subsidiary corporation, the Rapid Transit Subway Construction Company, which built parts of the Seventh Avenue line and the intricate junction between the Lexington and Park Avenue lines. Previous to his connection with subway work, he had been much in the public eye as head of the Panama Canal construction.

The new scheme of subways was a combination of old and new north and south lines. That is, the new Lexington Avenue line was made continuous with the old Park Avenue line; and its counterpart to the west, the new Seventh Avenue line, was made continuous with the old Broadway line. This left the old east-west connecting link through 42d Street, which was then converted into a shuttle servicing the two north and south mergers, in effect forming the crossbar of a huge H-shaped system.

In the opinion of many engineers, the separation of the two main parts was not sufficiently publicized in advance of the first operation. Utter confusion in the subway traffic reigned. In the late afternoon of that first day, when the great working population of New York City "homeward wound its weary way," many riders, as had been their habit, took trains on the East side of downtown Manhattan and upon reaching 42d Street endeavored to cross to the west side trains by means of the shuttle. The usual subway congestion was greatly increased and no one seemed to know which way to go. One passenger, who had evidently been imbibing too freely, appeared sincerely to believe that Mr. Shonts was personally responsible for the confusion. As he unsteadily climbed from the platform at 42d Street and Park Avenue, he was heard to remark, "It's a —— good thing they didn't let Shonts finish the Panama Canal."

Construction of engineering works often begets lawsuits, such as that I attended in the winter of 1907 at the Courthouse in Mineola, Long Island. Engineers, foremen, contractors, and lawyers gathered as witnesses or advocates. The dispute concerned a certain Virginia Hotel at Columbus Circle in Manhattan. Its owners sued the contractors, alleging that the subway work had interfered with its business and caused great losses and damages. One of the defense lawyers was an elderly man, of solemn mien clad in a long black square-cut coat. His beard was long and white, and when he looked up from his documents to address the Court he threw back his head so that the beard stood out in front of him almost horizontally. To look at him one would never have assumed that he possessed a spark of humor.

All through the evidence, especially in the cross examination by the contractor's counsel, it had been intimated that the guests of the hotel in question were not of the highest character, in fact that they were people whose morals were definitely below par. Finally a witness, speaking in broken English, testified that "every time the blast goes off a man with a red flag comes in and chases out the guests." At this point the stern-looking attorney for the defense arose and solemnly addressed the Court somewhat as follows: "Your Honor," he said, "I would like to call to your attention that this building was a hotel and therefore in the nature of a public place. For which reason the man with the red flag had a perfect right to enter, in fact it was his duty to so enter and consequently, Your Honor, it makes no difference whether the guests were chased or unchaste." This remark, coming from such a solemn looking individual, brought the trial uproariously to a temporary halt.

A PUZZLE SOLVED

In 1909 it was decided to interpolate a new station on the Broadway line at 191st Street, between the 181st and Dyckman Street Stations. The railroad at this point is in a deep tunnel through the bedrock of Manhattan. Excavation necessitated a supply of compressed air, and in order that the excavation of the platforms and a

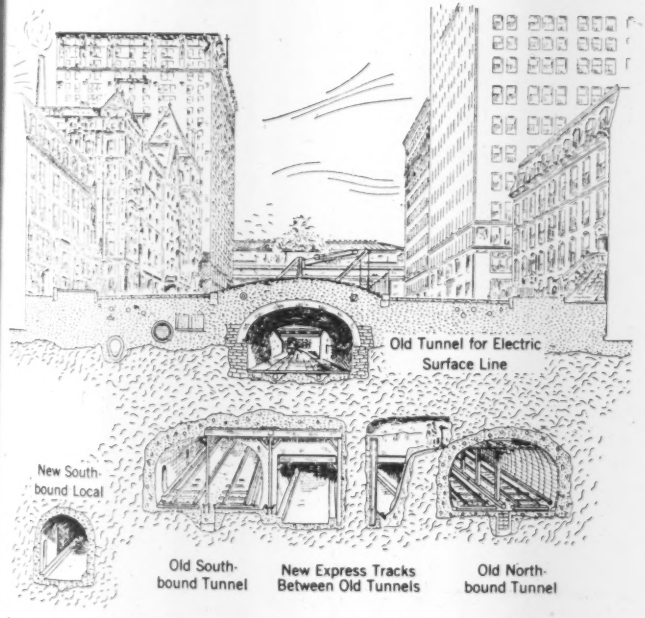
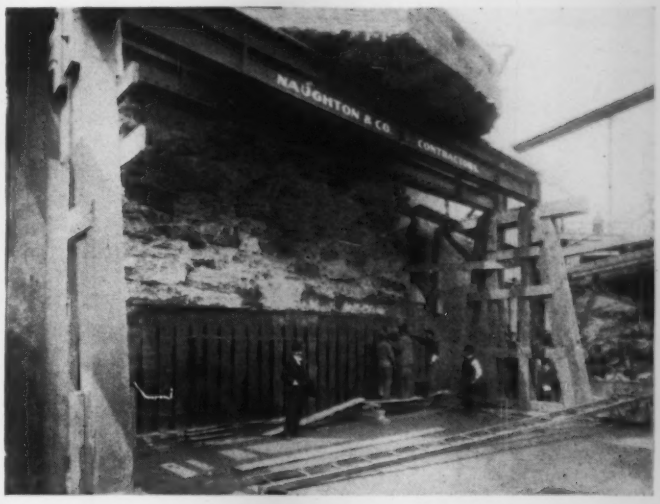

Old Tunnel for Electric Surface Line
New South-bound Local
Old South-bound Tunnel
New Express Tracks Between Old Tunnels
Old North-bound Tunnel

FIG. 2. CROSS SECTION OF PARK AVENUE, LOOKING NORTH FROM NEAR 39TH STREET

First Bent of New Steel Shown in Place in Center; Excavation in Progress Over Old Northbound Tunnel (Right) (From TRANSACTIONS, Vol. 82, 1918, Page 303)



Board of Transportation, N.Y.C.

SUPPORT FOR COLUMBUS MONUMENT, IN CENTER OF COLUMBUS CIRCLE, 59TH STREET AND BROADWAY (ORIGINAL SUBWAY) WHEN WATERPROOFING WAS UNDER WAY

The Engineer and Even the Inspector Wore a Derby Hat. It Was Excavation near This Point That Caused Interesting Litigation

long passageway might proceed simultaneously with the shaft, a means had to be found by which compressed air could be delivered into the tunnel at 191st Street as well as on the surface. It was determined to make a boring from the surface with a shot drill and to place an air line in this boring. For fear that the core from the lower part of the boring might drop on the tracks and cause a derailment or worse, it was requested that the boring be located just inside the west wall of the existing railroad tunnel.

In due course the boring reached the depth of the tracks, but for reasons then unknown, it did not enter the tunnel. The Superintendent intimated that our location had been incorrect and I had a dose of "cold feet"; but a check proved that we were correct. For a time matters were at a standstill. Mr. Ridgway was then with the Board of Water Supply and knowing that he had made many core borings on the center line of the Catskill Aqueduct, I hastened to ask him if all of his borings entered the cross section of the Aqueduct tunnel. He told me that while some of them were found when the tunnel was excavated, many of them never were. From this I was convinced that our boring had "gone out of plumb" and was outside of our tunnel wall. After considerable discussion I persuaded the Superintendent to explode some dynamite in the boring late at night when there was a long interval between trains. When this was done a crack appeared in the tunnel wall. This proved our theory to be correct. A small compressor taking current from the third rail was set up, and the tunnel wall and the rock behind it were removed. The boring was found about 8 ft outside the tunnel wall, whereas it had started on a line 6 in. inside it.

ALMOST A STALEMATE

One of the difficult pieces of construction in combining the Broadway and Seventh Avenue lines, previously mentioned, was at Times Square, the junction point. The work made good progress but it was evident that cooperation between the contractors and the operators of the existing subway was lacking. This was especially noticeable when the time came for the operating company to undertake certain work for the contractor. It finally resulted in a conference in the office of the chair-

Board of Transportation, N.Y.C.

OLD GRAND UNION HOTEL, PARK AVENUE FRONT

This Hostelry Was a Landmark. Between It and Grand Central Station, at Left, Appears Station of Elevated Line, Subsequently Removed. Elevated Road at This Point Was Reputedly Subject to Aerial Bombardment from Subway Blasting

man of the Public Service Commission. The "higher-ups" were present in full force. The talk opened with a statement on the part of the operators that only the previous night the contractor had dropped a piece of concrete on the third rail protection board and broken it. Chairman McCall, evidently thinking this was a major catastrophe, swung around in his swivel chair and glowered at me. "Yes," said I, "and it was only very recently that this operator constructing similar subway work at Park Avenue and 42d Street, shot off a blast and threw a stone through the window of his own Car No. 13 on the 42d Street Shuttle of the 3d Avenue L. And he did not consider it necessary to write the Public Service Commission any letters about that!"

Conversation came to what in railroad parlance would be termed an emergency stop. The silence was actually so dense that it could have been cut. I think this retort made all concerned realize that the accidents referred to were relatively unimportant matters and that it was no time for fault finding or child's play; that we were all doing our part in a serious undertaking and that we must all work together. In any event the result indicated just that, and before the conference ended, a "modus operandi" had been determined and the work at Times Square went on to completion.

A PROBLEM IN ALINEMENT

Connecting the new Lexington Avenue line to the original tunnels under Park Avenue was also a complicated business. In his autobiography (CIVIL ENGINEERING for January, February, March, and April 1939) George H. Pegram, Past-President of the Society, referred to it as "the most difficult piece of construction in the world." As the new line turned diagonally from Lexington Avenue (Fig. 1), it passed under the Commodore Hotel, across 42d Street and the site of the Grand Union Hotel, and so to the Park Avenue connection. Part of the southbound local track was in tunnel. It extended under both of the existing tunnels, then between the southbound one and the westerly building line of Park Avenue.

After the tunneling had progressed diagonally south, under and to the west of the existing railroad, a blast one day revealed metal rods projecting from the rock roof. The tunnel men began to wonder if they had the "jitters" but the phenomenon had a perfectly good explanation. When the excavation was being made for the adjacent Belmont Hotel, which was built concurrently with the first subway, the rock had a tendency to slide into the cellar excavation from the east or Park Avenue side, and to prevent this the excavators drilled long holes in the rock and placed steel rods in these holes or, as the expression among rock excavators is, "pinned" it. The "pins" happened to be long enough to reach into what was later to become the bore of the southbound local track.

A little later this bore "side-swiped" some brick masonry; more "jitters," but it proved to be only the wall of the wine cellar of the Hotel Belmont. By this time, the tunnel had about rounded the curve and was presumably proceeding southward on its proper course along Park Avenue in front of the Murray Hill Hotel. My Holland ancestry asserted itself and in my conservatism I began to wonder if the tunnel was really where it belonged. I had the assistant engineer in charge of the work, who was an ultraconservative, remeasure the total deflection around the curve. Everything was found in order.

Meanwhile the drills and the dynamite were at work and the tunnel bore was lengthening. If it were not on its correct line it was approaching or running away from the existing tunnel (Fig. 2). Either of these courses would be disastrous, as one way it would encounter moving trains and the other would encroach on private property. The thought would not leave me, so I finally arranged to have a horizontal hole drilled through the intervening rock. This done, a steel tape was rodded through the hole, and the distance between tunnel centers was found to be correct. So I dismissed this worry from my mind, but no doubt within a short time acquired others which bothered me as much or more.

On the site of the old Grand Union Hotel, at the southeast corner of 42d Street and Park Avenue, now stand large office buildings supported by the subway roof. The subway structure was designed to carry the load of such buildings and in constructing its foundations we exercised what we thought was due care in seeing that all unsound rock was removed. As a result the contractor accused us of "manicuring" the rock and of causing a cold draft to impinge on the Devil in the hot place below.

Probably every man regrets some of the things in which he has had a part. I know I do. One of them is the razing of the old Grand Union Hotel. I never made use of it as a hotel but on many of my working days I patronized its renowned lunch counter, and in fact I had my lunch there on the day of its closing. I suppose its removal was a necessity but the hotel was an "institution" and it is one of my regrets that it had to close. Nothing to my mind ever quite took its place.

Bridge and Culvert Flow Areas

Current Practice in Determining Runoff

By F. William Greve

Professor of Hydraulic Engineering, Purdue University, Lafayette, Ind.

Portland Cement Assoc.

Quadruple Box Culvert on the Natchez Trace, Jackson, Miss.

In many areas rainfall and runoff records are woefully incomplete. Therefore the application of empirical formulas sometimes complicates rather than assists in the determination of design quantities. Several procedures that have been proved reliable are discussed by Professor Greve in this paper, which was delivered before the Twenty-Ninth Annual Purdue Road School.

Determination of the maximum flood flow that may pass down a stream depends on its entire past history, which unfortunately is not a matter of record. How often one hears statements to the effect that distinct climatic changes are taking place within the relatively short span of a lifetime, and that rain storms and stream flows are higher, lower, or both, than they have been in the past. Recorded data, on the other hand, give evidence that there has been no distinct climatic change in this country during approximately the last two centuries. Undoubtedly there has been some change in the frequency and intensity of minor floods, caused by man-made alterations in the vegetal cover of the earth's surface, construction of flood planes, and construction projects of assorted types. Major floods, on the other hand, will occur, man's work to the contrary, whenever abnormal precipitation falls on ground that is frozen or saturated.

It behooves engineers not to accept as fact the memories of the older generation with regard to stream records. Almost everyone can remember the brooks and rivers of his boyhood fishing days. Their flow has become magnified with the lapse of time, like the fish that inhabited them. These were giants, so it seems, compared with the pigmies of today. Such memories furnish some of the spice of life and provide source material for good-natured bragging. But if one would preserve these illusions, let him shun the streams of his youth, and under no circumstances permit his technical judgment to be swayed by recollection.

Factors affecting runoff from small watersheds include: (1) form, intensity, and duration of precipitation; (2) season of the year; (3) type of vegetal cover; (4) type of soil; (5) topography; and (6) percentage of the surface occupied by natural storage basins in the form of ponds and marshes. In the study of surface flow from large areas, it is necessary to include such additional hydrological factors as direction of the storm, size and shape of the watershed, seepage, evaporation, and synchronization of tributary flow.

Sectional Steel-Plate Arches for a Small Bridge in Washington County, Indiana

There are several methods for the estimation of runoff, most of them of the rule-of-thumb, or empirical order, and frequently quite suitable, for the purpose at hand. Most highway and railway engineering staffs have a generous supply of such empirical formulas on tap, each one limited in application to a restricted region of the country. A brief description of the methods follows:

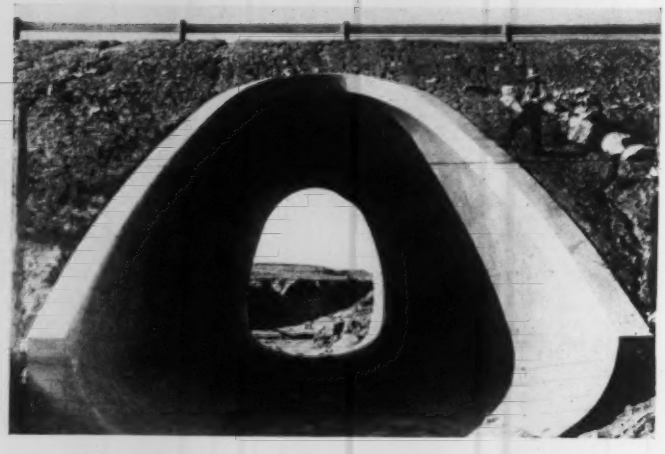
1. The flow can be estimated by comparison with that of a similar stream in the same locality for which data have been obtained by measurement.

2. For a stream that is at all times confined within its banks, the cross-sectional area of the waterway corresponding to maximum peak flow can be approximated. This approximation, multiplied by a presumed velocity of flow equal to 10 ft per sec, will give the probable flood discharge. Clues to the maximum height of the stream surface frequently can be secured from cornstalks and other debris lodged in trees along the banks.

3. A method often used applies one of many empirical formulas of the general type, $Q = C\,R\,D$, where Q is the discharge in cu ft per sec, C is a variable coefficient dependent upon the physical characteristics of the watershed, R is the total rainfall in inches, and D is the area of the drainage basin in square miles. The accuracy of the results will depend very largely upon how well the engineer estimates, or judges, the value of C. (It is, psychologically, bad practice for an engineer to admit that he ever guesses.) Probably no two engineers would evaluate C alike for any one watershed; and it is readily understood that values of C obtained from data established on one drainage basin should be applied with extreme caution to other watersheds in a different area. Equations of the general empirical form mentioned have the further handicap that no factor is included to express the intensity of the rainfall, which may be and often is of paramount importance.

Some of the better-known formulas include Talbot's, Peck's, Myer's, and the Tidewater. Talbot's formula is based on data collected from areas of less than 50,000 acres in the Mississippi Valley. The following examples illustrate the wide variation in results obtained by applying several of these formulas to two watersheds in California. Myer's formula applied to a watershed of 100 acres indicated a flood flow 2.5 times that computed with the Tidewater equation; for an area of 100,000 acres, the maximum runoff calculated with the Peck formula was 8.5, 15, and 53 times, respectively, that computed by the Tidewater, Talbot, and Myer equations.

4. The so-called rational method for computing run-



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PARABOLIC ARCH CULVERT, BARBER COUNTY, KANSAS

off takes into consideration both the intensity and duration of the rainfall, the elapsed time for water to reach a given point on the stream from various parts of the watershed, and the ratio of runoff to rainfall. The total rate of discharge is then computed from a combination of these several factors. H. K. Barrows, M. Am. Soc. C.E., states that, "usually surface flow on lateral slopes will move with a velocity of from 0.50 to 1 mile an hour; on small branches the velocity may be from 2 to 4 miles an hour, depending on the amount of fall." He also states that "the percentage of rainfall flowing off under flood conditions is likely to be from 50% to 75%, although with conditions of sandy soil and flat slopes this may be considerably less." A maximum ratio of runoff to rainfall equal to 75% seems too low, for it is conceivable that the runoff may equal or even slightly exceed the precipitation if a concentrated warm rain falls on a rough terrain covered with deep snow. The stream flow in such an instance is determined not by the precipitation alone, but by the combined rainfall and melted snow. Discharge determined by the rational method is usually calculated by substitution in an equation of the form, $Q = C\,I\,A$, where Q is in cu ft per sec; C represents the ratio of runoff to rainfall, sometimes termed the runoff coefficient; I is the intensity of rainfall; and A is the area of the watershed.

5. Accurate measurement of the stream flow can be made with a current meter if a suitable metering station is available. The first step in the procedure is to divide the vertical cross section of the stream into component areas. The mean velocity in each component area or section is then obtained by manipulating the meter in one of several ways—by (*a*) moving the meter slowly and at a uniform speed from the water surface to the bottom and up again, the mean velocity being obtained by integration of the meter readings; (*b*) averaging the velocities measured at given intervals of depth; (*c*) assuming the mean velocity equal to the average of the velocities measured at 0.20 and 0.80 of the depth; and (*d*) assuming the mean velocity to occur at 0.60 of the depth. The number and size of the component sections must be decided by the judgment of the engineer, which will be guided by the relative regularity of the stream bed. The integrating method is seldom used because of the difficulty of maintaining uniform speed of movement. The multiple point method is dependable, but its reliability is influenced to a large extent by the lengths of the intervals employed. This method will require more time than any of the other three. The 0.2–0.8 method is preferred by most engineers; the 0.6 method will lead to approximate results only.

6. Reliable data of discharge are obtainable with measurements made at a weir, or dam, placed across the stream, the rate of flow being determined by the size and shape of the weir and by the depth of water over or through the structure.

All these methods, except the two that make use of the current meter or the weir, require measurement or else close approximation of the rainfall. The number of rainfall gages is being increased yearly, but the time is far distant when there will be enough of them for hydrological purposes. And until such time arrives, the correlation between stream flow and precipitation must rest in large measure on the engineer's judgment.

Up until about a decade ago, many if not most engineers assumed that a peak flood flow of 600 cu ft per sec per sq mile of drainage area was adequate for design purposes in the central section of the Mississippi Valley. At the present time, however, an increasing number of the profession believe that hydraulic structures on small streams should be capable of handling a flood of from 2,000 to 2,500 cu ft per sec per sq mile. One approach to the problem is to suppose that any given area may be subjected to the greatest storm that has ever occurred on any similar basin in the same region. Assuming that runoff will equal rainfall will lead to conservative results, which will be more reliable than can be obtained from a study of a stream's past record, mainly because data for rainfall are more numerous and generally more dependable than are those for stream flow.

OBSERVATIONS ON CULVERT DESIGN

Many designers of culverts and small bridges compute the required cross-sectional areas of the waterway by empirical formulas similar to the expressions previously discussed for computing runoff. Both sets of equations possess about the same advantages and disadvantages. The rational approach to the problem is to treat the culvert as a short pipe, or tube, flowing full at peak discharge. It can be easily proved, by applying the Bernoulli theorem, that $q = AC\sqrt{2gh}$, where q is the discharge in cu ft per sec; A is the cross-sectional area in sq ft; C is the coefficient of discharge, determined experimentally; g is equal to 32.2 ft per sec per sec; and h, the head, is the difference in elevation in feet between the water surfaces at inlet and outlet. The value of c will be influenced by the shape, cross-sectional area, and length of the culvert; the material of the culvert; the head; form of entrance and outlet; and depth of submergence. Values of C are included in various handbooks, notably King's *Handbook of Hydraulics*. More detailed information is available in the pamphlet, *Flow of Water Through Culverts*, by Yarnell, Nagler, and Woodward, published by the State University of Iowa.

The prime prerequisite in the design of a culvert is to provide sufficient hydraulic capacity at all times. Failure to do so may be serious. In the final analysis, the engineer will have to make decisions based on his own experience, quite apart from mathematical applications.

Portland Cement Assoc.

REINFORCED CONCRETE BRIDGE ON THE NATCHEZ TRACE, JACKSON, MISS.

The Materials-Economics of War Housing

Standardized Designs, Avoiding Critical Materials, Characterize New Workers' Homes

By JACOB L. CRANE, JR., M. AM. SOC. C.E.

DIRECTOR OF URBAN STUDIES, OFFICE OF THE ADMINISTRATOR, NATIONAL HOUSING AGENCY, WASHINGTON, D.C.

MUCH ingenuity in design and construction has been shown in the development of war housing. New fabrication processes and new materials are being used extensively. Because of the material shortage, in several cases the effect has not been to raise the standards of housing facilities. Often it has been necessary to provide accommodations at minimum standards or be faced with the problem of tent cities. This paper by Mr. Crane was presented before the City Planning Division at the Society's Annual Meeting in New York.

WAR housing involves a new kind of economics. While there are still statutory and administrative limits on the dollar cost of war housing per family accommodated, while the dollar cost can still be computed per square foot or per cubic foot, and while the monthly costs are computed in determining rents, as a basic control dollar-economics has given way to materials-economics.

By definition, the demand of the war program for certain materials requires that as much of them as possible be made available for production of aircraft, ammunition, tanks, and ships. Consequently it is necessary to use for war housing as little of these materials as can be managed. Not dollars per family dwelling unit, but pounds of metal, constitutes the standard by which the economics of war housing is measured. This new standard produces a type of housing program unique in the United States.

War housing is also affected by other controlling factors. The minimum of manpower should be used in construction, and this has stimulated both the off-site and the on-site fabrication of the elements of housing, and in fact of complete houses. Additional load on transportation facilities for the movement of materials must be avoided, and this factor has created a tendency toward the use of local materials and of factory fabrication at points as near the housing sites as can be arranged. Above all, speed is essential, and heroic efforts are made to discover the firm needs for war housing as early as possible and to cut down the time for site selection, design, and construction from a normal period of six or eight months to one ranging from a few weeks to three or four months. But the dominant factor in the economics of war housing now has to do with minimizing the use of materials which are needed for other war purposes.

These considerations taken together require that only indispensable war housing be programmed and built. Accordingly, the present policy proposes that during the emergency no new housing be erected to increase or improve the supply of shelter for persons who are already residents of any community. The entire national effort in the housing field has to be directed toward accommodating essential war workers and their families who are drawn from other localities into the areas of war production—that is, for those war workers and their families who might be termed "inmigrants." This policy interrupts the normal production of dwellings for replacement of old houses, that for natural increase of population and for newly formed families, and that for "inmigrants" other than war workers. Hence the present limitations unavoidably hasten the deterioration of the national housing supply, which in fact had already fallen behind because of the lean residential construction during the 1930's. But the war housing policy does serve very greatly to reduce the requirements for critical materials.

The magnitude of the current war housing job, under the present policy, can be visualized from the following figures. From July 1940 to July 1943 a total of nearly three million dwelling units of actual war housing has been required, that is, accommodations for "inmigrant" war workers. Of this total, about 1,200,000 are required for single persons, nearly 400,000 for two-person families, and about 1,300,000 for families of three or more persons. Out of the three million total, it is proposed to provide somewhat more than half in existing housing and somewhat less than half in new construction. Thus new

National Housing Agency

ONE OF THE EXPERIMENTAL HOUSES ERECTED TO DETERMINE METHODS OF CONSERVING LABOR AND MATERIALS

National Housing Agency

TWO-FAMILY DWELLING UNIT IN A WAR HOUSING PROJECT IN CONNECTICUT

National Housing Agency

LOOKING INTO A COURT IN A DORMITORY APARTMENT PROJECT AT PORTLAND, ORE.

Close-Knit Grouping Conserves Materials Needed for Utilities

residential construction for war housing purposes totals approximately $1\frac{1}{4}$ million units, built and to be built—as dormitories, two-person accommodations, and dwelling units for families of three or more. To reduce these requirements for new construction to the indispensable minimum, non-essential "immigration" is discouraged; the "immigration" of single workers and of married workers who leave their families at home is encouraged; and every reasonable effort is made to utilize fully the supply of existing housing.

In providing the indispensable new housing, the central problem is that of reducing the use of metals and, at certain periods, of some types of lumber to the feasible minimum. Sites are selected so far as possible where they are accessible to existing utilities on the one hand, and on the other where they will not require transportation facilities beyond those already available. The ideal site is one within walking distance of the war establishment (but not immediately adjacent to an important bombing objective), and served by existing water, sewer, gas, and electric services. Sometimes these factors are mutually competitive.

For example, the only available site served by utilities may require daily commutation by the use of passenger automobiles. Such a situation raises the problem of rubber for tires versus metals for utilities; and while both are to be avoided, if a choice must be made between the two it is considered sound policy to save on the metals for utilities even though this means the use of rubber for transportation. Of course it is desirable to select sites which will not require new schools or other community facilities, or heavy grading, and which will be suitable from the point of view of the relationship between the war housing project and the neighborhood.

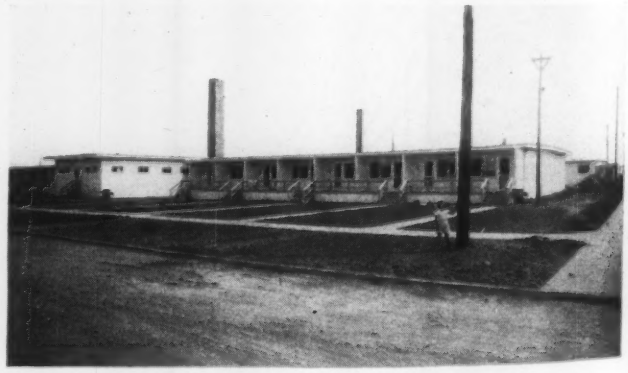
These and other factors make site selection complicated and, in a sense, abnormal; but the conservation of metals is controlling. Site plans must also be designed to minimize the use of metals. The War Production Board has established limitations on the utilities which may be installed for any war housing project, expressed in terms of lengths of pipe and so forth. These rules tend to produce compact projects, often of row houses.

Likewise, in the design of the buildings and their equipment, the National Housing Agency and the War Production Board have limited the use of metals and of certain types of lumber. Aluminum is not allowed at all; only the absolute minimum of copper wire; and practically no reinforcing steel, metal lath, metal sash, and such. Shower baths with very little metal are installed instead of bathtubs; "chimney furnaces" with little metal are coming into use in lieu of other types of heating and cooking equipment; ice boxes, shelves, and sinks are being made of non-metallic materials; and various types of wallboard and masonry have been used to conserve soft-wood lumber.

The materials-economics of war housing has developed a great deal of ingenuity in design and construction, and in the use of new materials and fabrication processes. And it has entirely changed the character of the housing. Whereas two years ago the typical little house built by private builders for defense workers was a detached cottage of fairly adequate standards, with normal pre-war equipment, and situated generally in a newly developed subdivision, the private war housing of 1943 has to be smaller, much of it in rows, on property already served by utilities, and with many substitutions for "shortage" metals and lumber. Altogether justifiably, privately built war housing represents somewhat lower standards than that prevailing before the war; and there may be a question as to the acceptability and use of some of this housing during the post-war period while it is still being amortized.

In its earlier stages, it seemed clear that government war housing, when built in localities where it could be used after the war, would be most economical if designed to permit its occupancy for twenty years or more. Accordingly in most of the built-up areas of defense production some permanent government war housing was built. Similarly, for the localities where this housing would not be needed after the war, it seemed clearly economical to provide demountable housing units. These could be moved after the war for use elsewhere. Some 50 thousand "standard" demountable single and twin houses were thus produced in this phase of the program, giving the "prefabrication" techniques their first large volume of work.

While these economic formulas still have some validity, the materials-economics consideration has forced the government program to become almost wholly temporary in character for dormitories, "dormitory apartments,"



National Housing Agency

MULTIPLE-FAMILY HOMES AT SENECA, ILL., PLANNED TO SERVE ONLY TEMPORARILY

and family dwelling units. All three categories are reduced to minimum floor areas, minimum use of metals for utilities and equipment, and (at least during most of 1942) minimum use of soft-wood lumber. The accommodations are small, more crowded on the site, and definitely temporary in character. This type of war housing not only economizes on metals but also reduces requirements for construction manpower, transportation of materials, fuel requirements, and time needed for getting it ready.

The weight of metals required per housing unit has been reduced by more than half. The 1940 "defense housing" project used from 4,000 to 9,000 lb of metals per family dwelling, for utilities, house construction, and equipment. The 1943 project uses something in the order of 2,000–3,500 lb of metals all told, per family dwelling, and of course very much less than that for the dormitory apartment and the dormitory.

MANPOWER-ECONOMICS LIKELY

This is where the materials-economics of war housing stands at present. If the war continues, we may find ourselves measuring the economics of this type of housing in terms of man-hours required per essential "inmigrant" war worker, since it it our national manpower which constitutes the basic resource for producing war equipment and for fighting the war. Of course, as materials and manpower are conserved in war housing, the dollar cost also tends to go down; but it is not the dollar-economy but the materials-economy and manpower-economy that count most during war.

There is no doubt that this period of war housing will vitally influence post-war programs. The construction these days of housing only for "inmigrant" war workers, and in large part only temporary, means that our national housing deficit is rapidly increasing and that there will be an enormous impounded demand at the end of the war. It appears that there will also be a huge volume of impounded savings, and of funds for investment, to meet this demand.

As matters now stand, we may expect a housing boom beginning soon after the war and continuing for at least a few years. Such a boom may be stimulated not only by the active "market" but quite possibly also by the new techniques which are being developed and which may make the ratio between family incomes and the cost of new housing quite different from that in the pre-war period. That is, using new materials and new methods developed during the war, new houses may be available at monthly rents or payments constituting a smaller proportion of the family income than the level of similar payments for old houses. If this happens, standards will rise above pre-war levels; and, if relatively full employment can be maintained and adequate guidance and control can be exercised, we will meet our housing deficit with interesting types of new houses.

National Housing Agency

DEMOUNTABLE FAMILY DWELLINGS AT VALLEJO, CALIF.
Foundations Adapted to Terrain to Minimize Grading

National Housing Agency

INTERIOR OF A FACTORY-FABRICATED DWELLING AT SENECA, ILL.
Essentials Only Provided

All told, the materials-economics of the period of war housing will profoundly affect the economy of our cities—through influences which are as yet difficult to evaluate as a whole or for individual urban communities.

Under private or government auspices, war housing is being built in some 450 localities. These range from large metropolitan districts to little rural communities where war establishments such as ordnance plants require the aggregation of hundreds or thousands of workers. The war housing in these various localities ranges from permanent dwellings, which it is believed can be utilized after the war, to demountable units which can be moved away, and strictly temporary housing which is expected to be demolished.

FUTURE USE UNCERTAIN

Of necessity, all this strictly war housing has been built primarily to serve emergency purposes. Even though the permanent houses are related to the future of the communities as best possible under the circumstances, in some cases they are located and developed with reference to war establishments of uncertain future use, and of course they are located so that they can be served by existing utilities and transportation facilities. In many cases they do not constitute elements of clearly established long-range city plans. The demountable and temporary housing, if and when removed, will leave behind many problems concerned with the use of the utilities, schools, and other facilities built to serve them.

In general the recent residential developments in these localities tend to deviate somewhat from pre-war patterns of land use. A great many of them have either been placed close in, to take advantage of existing utilities and transportation (which may turn out to be a good thing for the municipal economy), or they have been placed near outlying plants, starting or accelerating a suburban trend. Evidently post-war programs of housing and city building will be developed in the light of new problems and great new potentialities.

Native Cane Reinforcement in Concrete

Clemson College Tests Indicate a Limited Use of Abundant Material

By H. E. Glenn, M. Am. Soc. C.E.

Associate Professor of Civil Engineering, Clemson Agricultural College, Clemson, S.C.

BAMBOO has been used to a limited extent for concrete reinforcing in the islands of Japan. It was first employed by American engineers in the construction of the Cavite Naval Base in the Philippine Islands. Although never utilized to any extent in this country, native cane grows abundantly in nearly all of the southern states. To investigate its use as reinforcement the Claude W. Cress Research Fund financed the tests which are reported by Professor Glenn in this article.

IN an effort to study the possibilities of native South Carolina river cane, the Clemson College Engineering Experiment Station, through funds from the Claude W. Cress Research Fund, has been carrying on a series of experiments designed to establish:

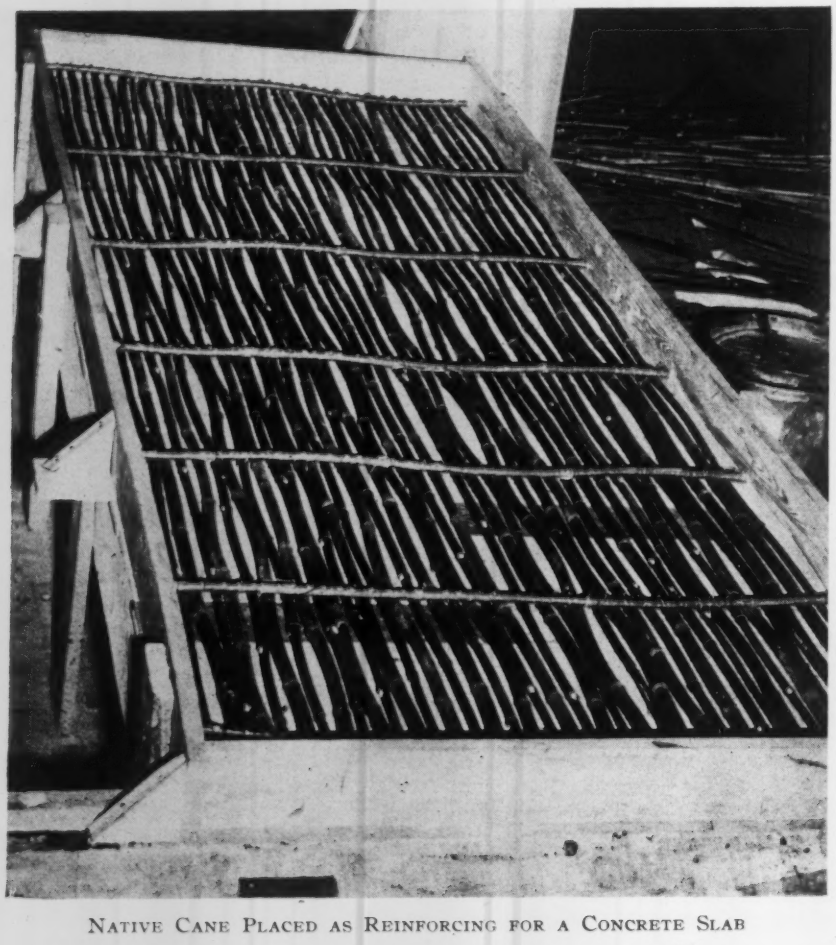
1. The physical characteristics of native cane.
2. The load-carrying limits of beams and slabs of cane-reinforced concrete.
3. Data to be used in the design of cane-reinforced concrete.

Three methods were used in determining the tensile strength of the cane and all of them gave reasonably close agreement in the results:

1. Individual pieces of cane with thick side walls were split and turned in a lathe so that a standard test piece at least 8 in. in length resulted, with the ends unturned to fit in the testing machine. The diameter of the 8-in. length averaged about $^3/_{16}$ in.

2. In the second series, pieces of split cane were carefully glued together and the whole turned in a lathe to give a standard test piece $^1/_2$ in. in diameter with a length of 8 in. turned, and the ends about $^3/_4$ in. in diameter.

3. The third series of data on tensile strength resulted from specimens prepared for pull-out tests to determine the bond between the concrete and the cane. Some of these specimens, where the diameter of the cane was small and the length of embedment long, broke in tension before they pulled out of the concrete.



Native Cane Placed as Reinforcing for a Concrete Slab

RESULTS OF TENSION TESTS

The average of all the tension tests showed that the tensile strength of cane was about 25,000 lb per sq in. on the net section. Individual specimens varied from 18,000 to 30,000 lb per sq in., but the majority of the specimens were in close agreement. There seemed to be no variation due to the seasoning of the cane. Some specimens were from green, new-cut samples and others were oven dried to constant weight.

Some factors noted in these tests were that the cane joints seemed to be the weakest point in tension, as any pieces having a joint always broke at that point. Also it was noted that the cane fibers pulled away from the joints from each end and never broke in the middle of a joint.

Readings were taken of the elongation under tension of the specimens used in the tension tests in determining the modulus of elasticity. In this case there was no noticeable variation between the green cane and the cured cane. The modulus of elasticity for the cane varied from 2,000,000 to 2,500,000.

Tests on bond were made by embedding one end of a 2-ft length in a 6- by 12-in. concrete cylinder. These specimens were tested by clamping the free end of the cane in the jaws of the testing machine and recording the load that caused bond failure. Here again both green cane and cured cane were used. Also green cane was cast in the concrete block and then dried before testing.

These tests gave relatively consistent results, which averaged as follows:

Green cane:

1. Big end of cane in concrete, bond 140 lb per sq in.
2. Small end of cane in concrete, bond 95 lb per sq in.

CLEMSON CADETS (COMBINED LOAD, TWO TONS) TEST STRENGTH OF CANE-REINFORCED SLAB

Cured cane:

1. Big end of cane in concrete, 190 lb per sq in.
2. Small end of cane in concrete, 150 lb per sq in.

Green cane when placed in concrete and then dried:

1. Big end in concrete, 110 lb per sq in.
2. Small end in concrete, 70 lb per sq in.

It was further noted that encasing a joint in concrete materially increased the bond stress.

In an effort to determine the change in diameter and volume due to drying, several specimens were subjected to oven temperatures of 110 C and dried to constant weight. The average of twelve such tests showed a volumetric change of slightly under 10% and a diameter change of 3.2%. This volumetric change looks dangerous, but when the fact is brought out that this change is largely on the inner fiber (hollow part) of the cane, its dangers are minimized greatly. There is a shallow layer of pithy material on the inner perimeter of the cane and this shallow layer shrinks greatly when dried. The diameter change of 3.2% is the significant factor here, and it has not proved dangerous in the actual tests which have been conducted in connection with this project.

A series of tests were made on cane-reinforced concrete beams $4\frac{1}{2}$ ft long, 4 in. wide, and 7 in. deep. In these beams, green cane, dried cane, and split cane were used as the reinforcing material. The results on the whole were rather uniform, there being little variation with the different methods of placing the reinforcement. Similar cross-sectional area of the reinforcement and load conditions gave similar results on green, cured, and split cane. When the percentage of cane was under 3 or 4% excessive deflection was noted. In most cases the load capacity of the beam was adversely affected.

Physical Characteristics of Native Cane

Tensile strength = 25,000 lb per sq in.
Modulus of elasticity = 2,000,000 to 2,500,000
Bond stress, big end embedded = 140 lb per sq in.
Bond stress, small end embedded = 60 lb per sq in.
Average bond stress = 100 lb per sq in.
Volumetric shrinkage, when dried = 10%
Diametric change, when dried = 3.2%

In all cases failure was due to shear cracks even when shear reinforcement was used. In some cases, shear cracks developed immediately above the reinforcement and followed a progressive horizontal crack throughout the length of the beam. In other cases typical diagonal cracks appeared, similar to those that occur in beams reinforced with steel.

In most cases final failure was due to localized failure in compression of the concrete under the load after diagonal cracks had developed the full depth of the beam. In no case was it noted that the lower covering of the concrete spalled off and allowed the beam to fail as is often the case in steel-reinforced beams. This is due to the stiffening action of the large percentage of cane in the beam.

TESTS OF REINFORCED CONCRETE BEAMS

When the cane was placed throughout the width and depth of the beam with a clear distance between the stalks of about 1 in., the beams failed at loads considerably lower. Failure was due to horizontal shear, the beam simply cracking on a horizontal line near its neutral axis. Evidently the effective area of the concrete along the neutral axis was considerably reduced by the cane, and as the horizontal shear was at a maximum there, failure was the result.

When using cane that has been thoroughly cured (dried) before placing, care must be taken if the beam is cured with water. If the concrete is immersed before considerable strength has developed, it will be cracked by the swelling of the cane. This means that there is a distinct disadvantage in using cured cane unless some waterproofing is first applied to the cane. Asphalt emulsion was successfully used to solve this problem, the cane being simply painted with a brush coat. However, it has been found that when green cane is used, the concrete can be cured in any manner and there will be no ill effects from swelling.

It was noted in these experiments that if the small branches or limbs that shoot out at the joints were not cut off, they aided materially as shear reinforcement. In all cases the cane should be carefully placed and anchored, and concentrated as closely as possible near the bottom of the beam. If the cane is not anchored in place, it will tend to float to the top of the concrete during placing.

TEST SLABS SATISFACTORY

In order to study the actual load conditions on a full-size slab, two slabs were constructed, one with whole

BEAM FAILURE DUE TO SHEAR
Large Percentage of Reinforcing Used Here Reduced Concrete Area Excessively

cane and the other with cane that had been split in half lengthwise. The span of the slab, center to center of bearings, was $11\frac{1}{2}$ ft; the width of the slab, 4 ft; and the depth overall, 5 in. The cane reinforcement, with an average diameter of approximately $\frac{1}{2}$ in., ran in the long direction, was spaced 1 in. on centers, and placed 1 in. above the lower face of the slab. The effective depth of the slab was 4 in. The uniform load and the central load were applied separately.

In one slab the cane was not split. Its area, per foot of width, was approximately 1.44 sq in. The cane was green, having been cut only the day before it was placed in the slab and the concrete poured. High early strength cement was used. One week after pouring the concrete tested 2,750 lb per sq in.

There was no apparent bond failure under the maximum concentrated load of 4,320 lb. Three cracks developed in the slab. The one at the center appeared first, and then those about $1\frac{1}{2}$ ft on each side of the center. The central crack developed progressively and was the cause of failure. There were no signs of local compressive failure at the top of the beam. Apparently deflection was caused by stretching of the cane and not by bond slippage, although some slippage could have occurred.

A second slab was similar to slab No. 1 in all respects with the exception that the cane was split in half along its axis. The area of cane, therefore, was only one-half of that in slab No. 1. The slab was made much better than slab No. 1, however. An excellent job was done in completely surrounding the cane with concrete. A concentrated load of 4,000 lb caused failure of the cane in tension.

Failure in both slabs was caused by excessive bending: in slab No. 2 the split cane deflected some 14 in. before failure occurred. This excessive bending without increasing loads would seem to indicate bond failure, but there were no visible indications of this when the slab was inspected after failure. There were no diagonal shear cracks in the slab near the supports, nor did the concrete spall off of the lower face of the beam. There was no evidence of compression failure in the slab, although this should have happened, as the calculated stress in the concrete was above its crushing strength.

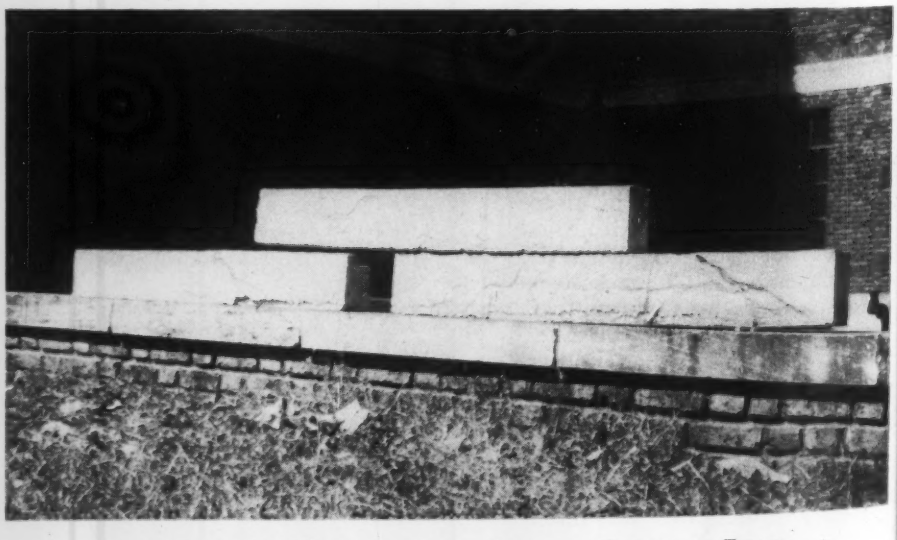
The interesting fact about these tests is the high loads the beams withstood without failure, and the remarkable amount of deflection at the center before failure resulted. With a 6- to 8-in. deflection, the slabs were still capable of withstanding considerable loads. This fact was also noted in the beam tests.

The bond stress in cane-reinforced members will always be of minor importance as it can easily be shown that the unit bond stress in properly designed members will be very low. This is because theoretically a cane-reinforced member will always break in compression in the concrete, the stress in the cane being very low for its actual area in contact with the concrete for bond. Seldom will the unit bond stress be over 20 to 30 lb per sq in. if the stress in the cane is under 4,000 lb per sq in. In none of the tests was bond failure observed before failure due to shear or to compression.

It seems to be fairly well proved that reinforcing of green undried cane definitely has sufficient bond resistance to prevent failure from this source, even though there is a fairly large reduction in diameter from the green to the cured stage.

In most of the beams tested, shear was the contributing cause of failure. When the unit horizontal shear exceeded some 70 to 80 lb per sq in., shear cracks usually developed unless shear reinforcement was provided. When such reinforcement was used, the unit shear was increased to from 100 to 125 lb per sq in. With more careful placing of the shear reinforcement, the value could be increased.

While the results of these experiments may not be absolutely conclusive, it has been shown that it is possible to construct load-carrying members of cane-reinforced concrete comparable to those that can be obtained in balanced steel design so long as the cross-sectional area of the cane is at least ten times that of the steel cross section. The permanency of cane as a reinforcing material is still not conclusively proved although previous experience in other investigations would indicate that there is no reason to expect that permanence will not be attained.



BEAMS IN ALL CASES FAILED IN SHEAR AS CRACKS IN THESE THREE SPECIMENS INDICATE

Engineers' Notebook

Ingenious Suggestions and Practical Data Useful in the Solution of a Variety of Engineering Problems

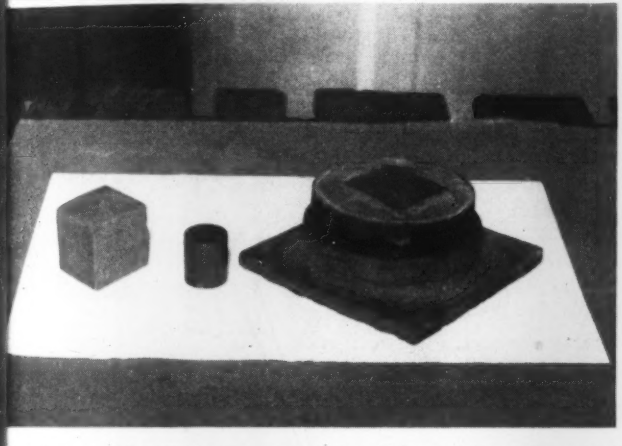
Load Required to Compress Concrete and Rock

By GEORGE B. SOWERS, M. AM. SOC. C.E.

CONSULTING ENGINEER, CLEVELAND, OHIO (NOW MAJOR, SPEC. RES., SCHOOL OF MILITARY GOVERNMENT, CHARLOTTESVILLE, VA.)

THE looseness with which engineers use the term "compressive strength" when applied to brittle or comminutable materials such as concrete and rock leads to erroneous assumptions and even to unnecessary use of expensive and vital materials. This is particularly true where structures are planned to rest upon high load-supporting piles and caissons.

Steel pipe piles $10^3/_4$ in. in outside diameter, supporting 70 tons of working load, will press on the concrete of the under side of the footing and develop pure compression in the footing concrete to the extent of 1,550 lb per sq in. directly over the pile head. Neglecting the bond between the outside of the steel pipe and the 4 to 12 in. of footing concrete in which it is embedded, pure compression is developed within the mass of footing concrete from 4 to 12 in. above its bottom surface.



TEST CUBE OF BEREA SANDSTONE, STEEL CYLINDER FOR APPLYING LOAD, AND A CUBE MOUNTED FOR TESTING SO AS TO PROVIDE FULL LATERAL SUPPORT

Likewise drilled-in-caissons 30 in. in diameter, without steel cores, which support 1,000 tons of working load, will press on the concrete of the under side of a foundation and develop pure compression in the footing concrete to the extent of 2,800 lb per sq in. directly over the end of the 30-in. diameter concrete-filled steel pipe of the caisson.

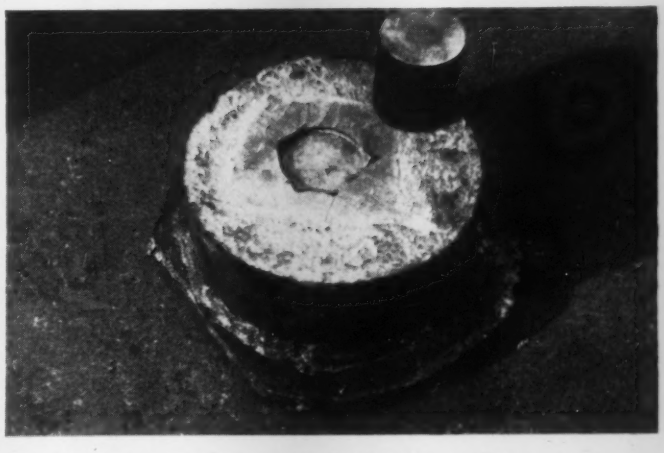
At what load per square inch will the concrete over the head of a high load-supporting pile or caisson be compressed or crushed? This question brought about a series of tests in connection with the building of a large ore dock and blast furnace foundations by the Western Foundation Company, for which the writer was serving as consultant.

The relation between pure compressive strength of concrete or rock and the "compressive strength," as shown by the crushing strength of cylinders and cubes, is not shown in textbooks or engineers' handbooks. The tables of "compressive strengths," or the usual laboratory reports of the "compressive strength" of concrete and rock, are in reality diagonal tension or shear rather than true compressive strength. The absence of data on pure compressive strength led the writer to make several comparative tests, all of which showed that the true compressive strength of concrete and rock is many times greater than the so-called "compressive strength," as shown by the crushing of cylinders and cubes. This relation could well be the subject of further tests as such brittle materials are not generally loaded in the manner directly simulated by the usual cylinder or cube crushing test.

An area $1^1/_4$ in. in diameter, on the surface of a laterally supported concrete block, sustained a load of 85,600 lb per sq in. before lateral flow took place, causing a disrupting of the surface around the loaded area. The concrete immediately below the steel block had been consolidated 0.15 in. Standard 6- by 12-in. cylinders of the same concrete crushed under an average load of 6,600 lb per sq in. Thus the true crushing strength of this concrete was over 13 times that of the cylinder crushing strength.

An area $1^1/_4$ in. in diameter, on the surface of a laterally supported Berea sandstone block, sustained a load of 60,000 lb per sq in. before lateral flow took place, disrupting the surface of the rock. The steel block penetrated the rock a distance about equal to the thickness of a piece of paper. Consolidation of the rock appeared to extend only about $^1/_4$ in. below the surface. The cube crushing strength of 2-in. sawed cubes of Berea sandstone was 8,000 lb per sq in. for an average of 5 cubes. Thus in the case of this fairly soft sandstone the true crushing strength of the rock was $7^1/_2$ times that of the cube crushing strength.

Three laterally supported blocks of 3,500-lb "compressive-strength" concrete similarly loaded with the $1^1/_4$-in. steel block each sustained a load of 25,000 lb per



SPECIMEN OF DEVONIAN SHALE AFTER FAILURE AT 11,650 LB PER SQ IN.

sq in. before there was any visible depression in the concrete surfaces. At loads between 60,000 and 65,000 lb per sq in., the steel block had penetrated the concrete block 0.2 in. with no disrupting of the surface around the loading block; the shape of this block prevented further penetration. The load-penetration curve indicated that the true crushing strength of the concrete would have been reached within another 5,000 lb per sq in. In these three tests it was found that the true crushing strength of the concrete was 18 to 20 times that of the cylinder crushing strength.

Careful observation of these three test blocks indicated that the penetration of the steel loading block began at a load per square inch of seven times the cylinder crushing strength.

The special expanding mortar used for mounting the stone specimens in the steel rings to provide lateral support had a cylinder crushing strength of 8,000 lb per sq in. This mortar when loaded over a $1^1/_4$-in. diameter area, as in the case of the concrete blocks, developed a true crushing strength of 40,700 lb per sq in. with a penetration of the steel block of 0.07 in.

A Devonian shale, which is the bedrock lying 120 to 175 ft below the Cuyahoga River at Cleveland, Ohio, developed a true crushing strength of 11,000 lb per sq in. when $1^1/_4$- and 2-in. diameter areas were loaded on the surface of laterally supported blocks. The 3-sq in. area showed a slightly higher strength than the 1-in. area.

The true crushing strength of the shale at 770 tons per sq ft, or that of the sandstone at over 4,000 tons per sq ft, provides very large factors of safety when the clean, sound rock socket of a drilled-in-caisson is used to support the foundations for bridges, buildings, and blast furnaces. Likewise, the true crushing strength of "3,500-lb" concrete of over 5,000 tons per sq ft provides a generous factor of safety in the concrete directly over a 70-ton pile or 1,000-ton drilled-in-caisson when used to support a foundation.

The behavior of the concrete test specimens laterally supported in steel rings shows that the true compressive strength of any area of concrete fully supported laterally is several times greater than the so-called "compressive strength," as now generally accepted and as developed by the customary cylinder test. This consideration is of vital importance in pile caps, and in other cases where only a portion of the concrete area is subjected to load. The Drilled-in-Caisson Corporation, which has specialized in foundation work, believes that full lateral support can be developed by especially designed reinforcing. The corporation expects to conduct further tests to confirm that belief and to produce the most efficient and economical form of such reinforcement.

Laminated Timber Bridge Deck Developed in Arizona

From a Paper Presented Before the Arizona Local Section

By R. A. HOFFMAN, M. AM. SOC. C.E.
BRIDGE ENGINEER, STATE HIGHWAY DEPARTMENT, PHOENIX, ARIZ.

SINCE all our timber is now going into essential construction for the war effort, the building of bridges of this material must be postponed. However, some of the recent developments in this field may prove of great value when highway construction again returns to normal.

Much has been published about the so-called "composite" timber and concrete deck. This design is described as a deck with a base composed of laminated pieces, usually 2 by 6 and 2 by 8, placed alternately and topped by a concrete slab which is thoroughly bonded to the timber base so that the two parts act as a unit in the "composite" section. Bonding is accomplished by many metal shear keys and spikes driven partially into the wood and allowed to project into the concrete top. In addition to the metal for bonding, an appreciable amount of reinforcement is required in the concrete slab, thus requiring more "critical" material than could be secured under the emergency program.

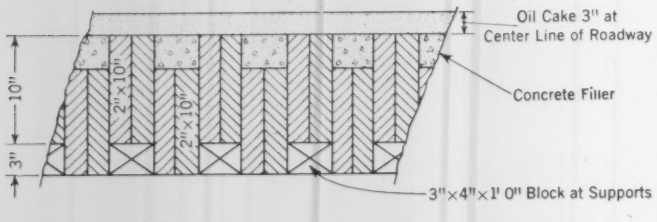


FIG. 1. SECTION OF LAMINATED TIMBER BRIDGE DECK

An alternative design proposed by the writer and developed by F. M. Shaw of the Bridge Division, Arizona State Highway Department, accomplishes the same effect of continuity without excessive dead load and without expending a large amount of critical material. Briefly, the design is composed of laminated pieces placed longitudinally with the span. The pieces are offset vertically to increase the depth of the slab. As shown in Fig. 1, they are offset in pairs to provide additional contact at the splices and to avoid having all splices occur in the same face of the slab where alternate pieces are spliced at the same point of the span.

It so happens that when two pieces of a laminated section are offset vertically, the resulting section loses strength (section modulus) until the offset equals one third of the depth of the piece, where it is again equal to the original section. The offset pieces, however, continue to gain in moment of inertia through the cycle even though losing in strength. The limit of offset is, of course, that point at which the remaining nailing space is just adequate for fastening the laminations together into a solid slab. Pieces 2 by 10 in., with a 3-in. offset, were adopted as approaching the ideal condition. Details of the curb have been worked out to provide additional strength for edge support.

A lean concrete filling, preferably a light cinder concrete, is placed in the channels in the top of the slab and broomed flush with the tops of the upper planks, in order to provide a firm base for the 2-in. asphaltic oil cake. This feature will prevent rutting, which would undoubtedly occur with a varying thickness of oil-cake surfacing. Laboratory tests on a model of this design have given sufficient proof that full continuity of the

structure can be developed with alternate pieces spliced at the quarter points of the span.

The proposed section is adequate for H-15 loading on continuous spans 24 ft long with end spans of $18^1/_2$ ft. Single spans should also be limited to a length of $18^1/_2$ ft. The material required is less than that needed for the standard type of 19-ft beam span with the same loading.

Stresses used in the design were 1,600 lb per sq in. in both tension and compression, with no allowance for the value of the concrete filling. Wheel distribution was taken as the wheel width of 15 in. plus the total depth of the slab, or a total width of $2^1/_2$ ft for each 12,000-lb wheel. Calculations for a 100% overload with a 50% increase in unit stress showed no overstress and only a $^3/_{16}$-in. deflection.

A series of laminated deck culverts, in spans of 6 to 10 ft, and in multiple spans up to four of 6 ft, 8 ft, and 10 ft, has also been developed, Fig. 2.

Pieces 2 by 8 in. were used in the laminations for all sizes. As shear stresses controlled on the shorter spans, no reduction could be made in the thickness of the laminated slab for the 6 and 8-ft spans.

The decks are supported by semi-gravity walls. These have no reinforcement whatever when under 5 ft high, and have only dowell bars in the construction joints when over this height. A water seal of asphalt and cotton fabric is provided over the entire deck to prevent seepage through it. No advantage was found in using offset laminations in these short spans as deflection is not a controlling factor.

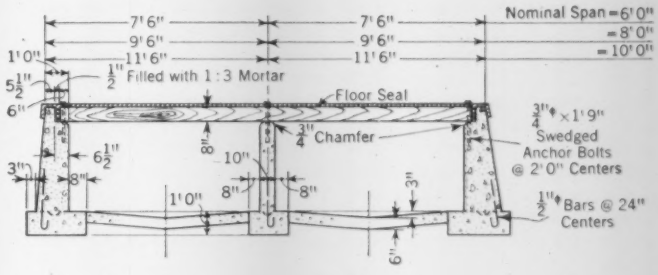


FIG. 2. CROSS SECTION OF TWO-SPAN TIMBER DECK CULVERT

Pressure-treated timber was contemplated for all types of laminated decks mentioned here. However, the few culverts built in Arizona so far have been of field-treated material.

Our Readers Say—

In Comment on Papers, Society Affairs, and Related Professional Interests

Treatment of Old Concrete for Binding with New

TO THE EDITOR: Some thirty-odd years ago, I discovered that I could get a perfect bond between new and old concrete by treating the old concrete with dilute HCl—two parts of water to one of acid. I clean the old surface as well as possible with brooms and water, then apply the dilute acid. The solution should be left for two or three minutes, and then washed off with clean water.

I gave this information to the profession many years ago, but have been surprised during the last year or two to find that very few engineers and constructors in general know about it.

WALTER F. SHAW, Assoc. M. Am. Soc. C.E.
Consulting Engineer

Jamestown, N.Y.

Historic Madeira-Mamoré Railroad Built by Americans

DEAR SIR: The letter on the Madeira-Mamoré Railroad by Edward D. Kingman, on page 187 of the April issue, is interesting history. However, Mr. Kingman omits the most important fact, which is that the railroad was actually built and put into operation by American engineers and contractors.

The concession for the construction of the railroad was obtained by Percival Farquhar, who raised the money. The company was incorporated in Portland, Me., although the money and management were pretty largely English and French. The late Percy H. Ashmead, M. Am. Soc. C.E., was chief engineer, and the contractors were Messrs. Mays and Jekyll. Surveys were started in 1907 before we had full advantage of the sanitation experience of the Panama Canal, but in spite of hardships and a good deal of sickness the line was opened to traffic a few years later.

FRED LAVIS, M. Am. Soc. C.E.
Consulting Engineer

New York, N.Y.

Solutions for Equations of Higher Powers

DEAR SIR: A comment on Mr. Didden's article, entitled "Practical Solution for Equations of Higher Powers," in the July issue, may be of interest to readers of CIVIL ENGINEERING.

The article is interesting but not new, as the author apparently suspected. My booklet, "Analytical Computations in Aerial Photogrammetry," makes extensive use of this same method extended to the case of three, and even to the case of five, simultaneous equations of higher degrees. Like Mr. Didden, I have often wondered why the method did not find wider use. My own applications of the method are also shown in "Analytical Computations in Photogrammetry," as published by the American Society of Photogrammetry in *Photogrammetric Engineering* for October 1941.

By the way, I believe the method originated with Newton.

EARL CHURCH, Assoc. M. Am. Soc. C.E.
Professor of Photogrammetry,
Syracuse University

Syracuse, N.Y.

DEAR SIR: In the July issue of CIVIL ENGINEERING Clement A. Didden describes a "Practical Solution for Equations of Higher Powers," which he says he has never seen used elsewhere. Undoubtedly many readers recognize the solution as Newton's method, which was first published in 1669.

It is still a useful method, but it is unsatisfactory in case the first derivative varies rapidly in the neighborhood of the root. Horner's method, which was published in 1819, is generally considered to be superior.

RALPH W. POWELL, M. Am. Soc. C.E.
Associate Professor of Mechanics,
Ohio State University

Columbus, Ohio

Comment on Herschel and Venturi

DEAR SIR: One may challenge Mr. Richardson's statement, on page 285 of the June issue of CIVIL ENGINEERING, that Venturi "never even dreamed that there was any relation between the rate of flow and the curious difference in pressure that he observed between the large and small ends of a cone discharging water."

Venturi was a professor of sciences, first at Modena and later at Pavia. He made many experiments on orifices, short tubes, and flaring tubes. He measured the flow, so he must have observed differences in head, and it stands to reason that he might have observed a relation between flow and pressure difference even though he may not have recognized its practical importance.

Clemens Herschel was a very fair-minded man. It is apparent that he must have recognized Venturi's priority of the principle involved or he would not have named his invention the Venturi Meter.

It was obviously impossible in the $1^1/_2$-page history of hydraulics in Manual 25 to mention every deserving contributor to the science. However, Clemens Herschel should have been mentioned in connection with the statement regarding the Venturi Meter. I have therefore asked the manager of the Society's publications to modify the text, if a second printing of Manual 25 is ordered, so as to read as follows:

"Giovanni B. Venturi, about 1784, made experiments with flaring tubes which disclosed the principle of the flow meter invented by Clemens Herschel and named by him the 'Venturi Meter.'"

J. C. STEVENS, M. Am. Soc. C.E.

Portland, Ore.

Independent Bargaining Group Formed

TO THE EDITOR: The article, "Engineers Protest Affiliation with Sub-Professionals," in the July issue, should be of interest to most professionals; for if our democracy is to survive, the prestige, dignity and importance of each individual must be recognized.

The second paragraph of the article indicates that a branch of the International Federation of Technical Engineers, Architects and Draftsmen's Unions conferred with the employer without certification by the National Labor Relations Board as bargaining representatives, and only after failure to agree was an election held. This was contrary to usual procedure. However, an election was held, and as some professional employees refrained from voting the case of the professionals was weakened. The War Labor Board awarded bargaining rights to Local 30, A. F. of L.

The Organized Professional Engineers and Architects of the Sunflower Ordnance Works was formed. This organization consisted of professional engineers, architects, high executives, and sub-professionals. Legal aid was obtained and a brief was prepared, stating objections to sub-professionals who would probably return to their former occupations as soon as the war effort was over. It is true that during the period of rapid expansion of many activities in engineering, those directing it have permitted, and even encouraged, thousands of meagerly equipped persons to enter the sub-professional ranks.

The brief states that those men will return to normal occupations. This statement probably weakens our case, for future predictions are not facts. These sub-professionals may stay with us, may even acquire the necessary knowledge to be classified as professionals.

It seems to me that the organization should be restricted to registered engineers and architects or else thrown open to all technical craftsmen; with the latter, various grades could be established within the organization.

The following case may be of interest to professionals and sub-professionals. In July 1941, the Federation of Architects, Engineers, Chemists and Technicians, C.I.O., attempted to organize Smith, Hinchman and Grylls, Inc., and Giffels and Valet, Inc., both companies of architects and engineers in Detroit. To counter, "The American Designers' Association" was organized by employees of both companies, with "no dues, no strikes, and no closed shops." The individual cash outlay for life membership was $1.50, which paid attorney's fees. Members were in all ranges from the lowest tracers to the highest registered professionals. Without dues there could be no mercenary activity; without closed shops each individual was free to make his own decisions; without strikes and with the tolerance of negotiation there could be no danger of disruption of war-effort production by members.

The F.A.E.C.T., C.I.O., was ruled out by the N.L.R.B., and the American Designers' Association prevailed. Harmonious conferences with management resulted beneficially to all concerned. The American Designers' Association is based upon the principle that the language of engineering is the only language that is incapable of misunderstanding and that the men who speak this language are the men who plan our industries. Who, then, of all men should be best capable of planning peace in industry? The engineer.

A similar organization enjoying similar results is the American Power Association, which experienced fruitful conferences with the General Motors Corporation.

Des Plaines, Ill. CHARLES F. PARKINSON, M. Am. Soc. C.E.

Forum on Professional Relations

CONDUCTED COLUMN OF HYPOTHETICAL QUESTIONS WITH ANSWERS BY DR. MEAD

For almost a year now Dr. Mead has been answering questions on engineering ethics in these columns. Currently he gives his views on Question No. 11, which was announced in the June issue. The question states that: "The chief designing engineer of a large company receives an urgent call from the chief engineer of another company, asking if he knows where an engineer can be found who would be qualified to do a certain type of work and who would be willing to work for $4,000 a year, the position being permanent. The only qualified person known to the chief designing engineer is a member of his own staff whose salary is $2,500 per year, who is regularly employed but who is not under contract for any specific period. The chief designing engineer replies that he knows of no qualified person, and does not mention the opening to the engineer on his staff for important work would be seriously retarded if he lost the man in question. Has he acted fairly and ethically?"

QUESTION No. 11, published in the June Forum on Professional Relations, was suggested in a letter by a young friend of the writer, who is employed in a department of a mid-West state and who apparently had experienced either directly, or indirectly through an associate, the treatment described from the head of the department under whom he worked. For that reason the question was not credited to its author. The answer to the question is quite clearly covered by the Golden Rule: "All things whatsoever ye would that men should do unto you, do ye even so to them."

There can be no doubt but that all men, employer or employee, would appreciate advancement either financial or otherwise regardless of its source, and they should so far as possible assist in the advancement of others.

In the writer's paper on "Standards of Professional Relations and Conduct," published in the TRANSACTIONS of the Society (vol. 106, page 22), this principle is stated: "The chief engineer or an engineer employing others in the profession owes to his assistants or employees proper consideration of their personal, financial, and professional welfare and advancement."

The writer believes that this should apply to all employees as

well as to engineers, and that it is one of the ethical principles most commonly disregarded. The writer has tried to observe this principle in his own practice and has helped his assistants to better positions and higher salaries than he was able to give even when replacing them involved considerable trouble and expense. He has also tried to aid them to a better understanding of the problems on which they were engaged and to give them as broad experience as possible in their office and field work.

It should be remembered that such problems are not usually so simple as the question seems to imply for the question assumes that the employee is able to fill the position in question. It is not always easy to judge other men's abilities; but an experienced chief engineer who has had charge of many assistants should be, and usually is, a good judge of the abilities and capabilities of the men who have worked under him for a long time.

On the other hand, few men are good judges of their own capacities and abilities, as is witnessed by the manifold failures in business and the poor work sometimes done on important projects. Young and ambitious engineers seldom think their abilities and merits are recognized as soon as they should be or that their advancement is ever as rapid as they deserve. Many ambitious young men would not hesitate to accept responsibilities that their chief would hardly be willing to trust to them.

For these reasons questions like No. 11 are sometimes quite impossible to answer without more knowledge than is really available. Nevertheless, the writer believes that in many cases the employees in engineering offices and in many other professions and businesses are not properly considered in these respects.

The writer is glad to quote three answers to Question No. 11 from three different branches of the profession. With the opinions expressed in these answers the writer is in essential accord. The first answer received was from C. R. Conkey, M. Am. Soc. C.E., vice-president and general manager of the Fegles Construction Company, Ltd., of Minneapolis, Minn. Mr. Conkey writes:

"The chief designing engineer who propounded Question No. 11 did not act fairly or ethically. Upon receiving the urgent call from the chief engineer of another company, and knowing that he had a man in his employ who would fill the specifications and who was receiving $1,500 less than the other company was offering, he should have in justice to his employee, and to the engineering profession, imparted this knowledge to his employee. He could have explained to him that because of the importance of the work in his office, he could not at that time release him. He should then have told the engineer from whom he received the inquiry, that he had in his employ a man who could fill the bill, but that due to the urgency of work under way he could not, at that time, release him. It would be only fair to the engineer employee, who had proved himself so valuable to the company, to give him an opportunity to accept or reject the offer of a permanent position at $4,000 per year; and if the work the engineer employee was performing was of so great importance that he could not be released, recognition of his services should be made by an increase in salary."

The second answer was from F. E. Turneaure, Hon. M. Am. Soc. C.E., dean emeritus of the engineering department of the University of Wisconsin, whose letter reads:

"Referring to Question No. 11 in the Forum on Professional Relations, it seems plain to the writer that the action reported was neither fair nor ethical. The employee should have been advised of the opportunity and the entire situation discussed with him. The engineer desiring help should also be informed regarding the qualifications of the employee and of the importance of the work in which he was then engaged. Frank and friendly discussion of the problem with respect to salary involved and probable future opportunities would probably lead to a satisfactory conclusion, either through an increase of salary or a deferment of change for a reasonable time.

"As dean of the college of engineering of the University of Wisconsin, the writer has advised in many cases of a similar nature, although with less difference in salaries than in the case mentioned; and the foregoing represents in general the policy of this college. A common case is that of a young instructor receiving an offer of a good job in engineering work which he desires to accept, if possible, although the offer comes in the midst of the school year. While we have no written contract with the staff, the appointment of instructors is generally for a school year. Nevertheless, it has been the practice of the various departments to release the man if practicable, and at any rate to do so at the end of the current semester.

"It seems to the writer that frank and honest dealings in such matters is of the greatest ethical value and will do much to enhance the reputation of an organization and to promote the loyalty of its employees."

The third answer is from an employee of a public engineering department who is, for obvious reasons, not further identified. Excerpts from the third letter follow:

"Question No. 11 on professional relations interests me very much, for I was once on the losing side of a similar situation. An offer was made to my employer for my services at a salary considerable higher than I was then or am now receiving. My employer refused the offer without mentioning the fact to me, neither did he raise my salary even a small part of the difference to justify (?) his action. I learned of the offer some months later when I happened to make the acquaintance of a man who had knowledge of the original offer. The action my employer took certainly did not raise him in my esteem.

"I would have appreciated being notified by my employer of the situation and being told why he was refusing my professional advancement. He probably justified his action in his own mind, and, then again, perhaps the matter never even entered his mind.

"Some chiefs of designing sections or engineering divisions apparently do not like to see their employees receive a substantial salary at an age younger than they themselves received a similar salary. It is a peculiar brand of selfishness which I and many others think is despicable. In our office several of our good men have been denied deserved salary increases and changes in ratings, because of age or personal dislike. And to add insult to injury, when another office makes an offer for the services of these men at a higher rating and salary, the office is nearly always told that the employee is not available.

"A typical reason for such action is the following: 'Well, we pay that man $3,800 and we can't raise him to $4,600 and change his rating, because all the rest of the men receiving $3,800 will expect some adjustment; and we can't let him transfer elsewhere at $4,600 because it would cost us a lot of money and time to fit a man to his job.'

"In specific answer to Question No. 11, I believe that the chief designing engineer has acted unfairly and unethically and that he was prompted to do so to maintain his record and to prevent retardation of important work. I believe that he should have notified his employee of the situation and thus prevented the occurrence of a distasteful situation at some later date when that employee by some means or other might learn of the offer. Most employees, I believe, can be prevailed upon to remain at their work until their assignment is complete, whether or not they have contracts. I believe that a mutually satisfactory arrangement could have been made among the three men concerned and that each would have maintained his self respect and the respect of the others and that everything would then have been both fair and ethical."

D. W. MEAD, Past-President and
Hon. M. Am. Soc. C.E.

Madison, Wis.

Other problems of professional relations will be treated by Dr. Mead each month. Next in sequence for study and written discussion by members until September 5, with answers in the October issue, will be the following:

QUESTION NO. 13: *A friend of Engineer "A," out of work and in need of employment, applied for a job in "B's" office, for which he was very poorly fitted. However, because of his fine personality he was being earnestly considered for the work. The job involved considerable responsibility, and the young enginer lacking this experience would, in "A's" opinion, need time to work successfully into the position. The applicant referred to "A," and the company has asked "A" to give his opinion of the man and his ability for the position for which he has applied. Should "A," with his knowledge of this applicant, advice the company concerning his real opinion of the man's qualification for the job, or should he so word his letter as to retain the friendship of the young engineer and let the company find for itself from experience the true worth of the young man seeking employment?*

SOCIETY AFFAIRS

Official and Semi-Official

Annual Convention Is Held in Los Angeles

THE Annual Convention being held in Los Angeles has been timed to take special considerations into account. It is therefore taking place much later in the month than is normal for such gatherings, and the meeting is still going on as the current (August) issue goes to press. Accordingly, it is hardly possible to give anything but a brief progress report.

In keeping with the stimulating technical program that has been prepared, a large group of members from the West Coast and the southwestern section of the country are gathered at the Biltmore Hotel. Social events, though necessarily curtailed because of the war, are being enthusiastically attended. The meeting has been designated a Convention in keeping with the constitutional requirements, and President Whitman is thus delivering the President's annual address.

Preceding the general and technical sessions, the Local Sections are holding their conference on Tuesday, July 27. In order to economize in time, The Board of Direction is convening on Wednesday afternoon, July 28, instead of meeting in advance of the general sessions as usual. In spite of all the restrictions naturally attending such gatherings in wartime, the meeting will add appreciably to civil engineering progress as well as to the war effort.

More complete details will appear in a later issue. It is also expected that a number of the technical papers will be printed in either CIVIL ENGINEERING or PROCEEDINGS.

Sunflower Case Placed Before War Labor Board

IN the previous issue of CIVIL ENGINEERING it was reported that a group of more than one hundred professional engineers and architects engaged on the construction of the Sunflower Ordnance Works at Eudora, Kans., appealed for the right to be represented by their own bargaining group and protested inclusion in a bargaining group known as Local No. 30, International Federation of Technical Engineers, Architects and Draftsmen's Unions, A. F. of L.

As related previously, testimony was submitted before the War Labor Panel by a committee representing the professional group, which is called the "Organized Professional Engineers and Architects of the Sunflower Ordnance Works." This committee had for its chairman Prof. W. C. McNown, of the University of Kansas, who is also an employee at the Sunflower project. Professor McNown had long been aware of the necessity for group action for the protection of employees, and he had repeatedly urged that the Kansas Engineering Society take leadership in this action. It was through his efforts that the professional employees of the Sunflower project were apprised of their rights. He was the only member of the American Society of Civil Engineers on the committee, and he advised it, organized it, prepared its brief, and otherwise directed its efforts. As chairman of the committee, he appeared repeatedly before the War Labor Panel.

Testimony was submitted also by representatives of Local No. 30, and these representatives objected to the testimony submitted by the committee for the professional group.

The War Labor Panel heard all the evidence from both sides of the case in formal session and reviewed the briefs submitted by both sides. Its report to the War Labor Board Region No. 7 recommended that the professional group be not allowed to intervene in the case on the grounds that this group was not organized and did not submit testimony until after Local No. 30 had been recognized as bargaining agent, and that the professional group did not adequately define its members. The War Labor Board Region No. 7 has the report of its panel before it for consideration and as yet (July 20) has not handed down its decision.

President Whitman Addresses Chinese Institute of Engineers

THE annual convention of the American Section of the Chinese Institute of Engineers was held at International House in New York City on June 26 and 27. Speakers at the convention banquet were E. B. Whitman, President of the American Society of Civil Engineers; Dr. Ping-Wen Tsou, Chinese delegate to the United Nations Food Conference; Dr. Ching-Chao Wu, secretary general, Ministry of Economic Affairs; and Mr. S. A. Trone, adviser to China Defense Supplies, Inc.

During the meeting of the Institute, whose 331 members in this country represent nearly every branch of engineering, technical sessions in civil, chemical, metallurgical, aeronautical, mechanical, and electrical engineering were held. The civil engineers discussed the design and construction of the "Detroit Industrial Expressway," a paper on this subject being presented by A. C. C. Chang.

ANNUAL CONVENTION BANQUET OF THE CHINESE INSTITUTE OF ENGINEERS

News of Local Sections

Recent Activities

ARIZONA SECTION

The Arizona Section held its spring meeting on June 8. Following a business session in the morning, there was a joint luncheon with the Student Chapter at the University of Arizona. During the luncheon it was announced that William Adams, Student Chapter president, had been selected as the recipient of the Section's prize of Junior membership in the Society. Since Mr. Adams has entered the armed services, the presentation was made in absentia. Another feature of the luncheon was a talk by Arizona State Senator W. F. Kimball, who spoke on "Legislative Aspects of the Colorado River." During the afternoon session papers were read by W. R. Lawrence, manager of the Tucson Division of the Consolidated Aircraft Corporation, whose paper was entitled "Consolidated Aircraft Modification Work and Its Relation to Wartime Needs"; and James H. Nelson, observer in charge of the Tucson Magnetic Observatory of the U.S. Coast and Geodetic Survey, who discussed "Observatory Methods of Obtaining Magnetic and Seismological Data."

BUFFALO SECTION

The June meeting of the Section got off to a good start with a steak dinner. After the members had done justice to this unusual treat, they heard Norman M. Herthe, president of the Section, discuss recent legislation affecting the engineering profession. The meeting was then turned over to Nelson Stone who, with the cooperation of the Bethlehem Steel Company, had arranged an interesting program. Two reels of sound films, entitled "Steel for the Armed Services," were shown. These covered all phases of the manufacture of steel from the mines to the finished product, with special stress on the scientific control of heat during the manufacture and proportioning of alloys. Later Eugene A. Gietzen, of the metallurgical department of the Bethlehem Steel Company, discussed the films.

CENTRAL ILLINOIS SECTION

On May 27 members of the Central Illinois Section heard Wilbur M. Wilson, research professor of civil engineering at the University of Illinois, speak on the shock-absorbing capacity of steel structural members as determined by drop tests. Engineering opinion has been divided as to the effect of high-speed loading upon the ductility of steel. Dr. Wilson, who has been experimenting on the problem for several years, showed by means of charts and diagrams that the stress-strain relation was not affected by speed of loading. Another feature of the program was the presentation of the Section's annual awards of Junior membership in the Society to outstanding Student Chapter members. The students thus honored were I. Sterling Snyder, Miss Marianna M. Schroeder, and Erwin F. Mueller.

DISTRICT OF COLUMBIA SECTION

"Recent Developments on the Inter-American Highway" was the subject of discussion at the June meeting of the Section, the speaker being E. W. James. Mr. James, who is chief of the Inter-American Regional Office of the Public Roads Administration, has been in close touch with the project from its inception. The war and resulting difficulties of water-borne traffic between the United States and the countries to the south have demonstrated the importance of an inter-American highway, he stated, and as a consequence work is being rushed on the uncompleted portions of the highway with the object of finishing it at the earliest possible date.

GEORGIA SECTION

On June 4 members of the Section elected Paul Weir as their president for the coming year; James C. Prather, vice-president; and M. T. Thomson, secretary-treasurer. During the inauguration of these officers it was pointed out that the Section should participate more actively in the civic affairs of the community, since engineers are naturally informed on city planning and community improvements. That evening about fifty members of the Section and their wives and friends held their semi-annual dinner dance at the American Legion Post in Piedmont Park.

ITHACA SECTION

At the May dinner meeting of the Ithaca Section, held on the Cornell campus on the 20th, the speaker was Dexter S. Kimball, dean of engineering, emeritus, of Cornell University. Dean Kimball, who has for some time been associated with the War Production Board, discussed the work of the Board.

KENTUCKY SECTION

The technical program at the June dinner meeting of the Section consisted of a talk by Carl Berg, engineer-secretary of the Louisville and Jefferson County Planning and Zoning Commission. Speaking on the topic, "Comprehensive Planning for Louisville and Jefferson County," Mr. Berg traced the history of city planning from ancient times to the modern period, including a comparison of current American and European practice. He then outlined the comprehensive project being carried out locally. The local board has originated and administered development plans for rivers and harbors, flood control, sanitation, airports and rail terminals, housing, and zoning.

MARYLAND SECTION

Since the June meeting of the Section was the final gathering before the summer recess, no technical program had been arranged. Instead the members devoted themselves to an evening of fun and good fellowship. The list of guests included A. M. Rawn, Director of the Society from California, who spoke briefly and informally, as did Director G. J. Requardt. The entertainment committee had arranged a nine-act floor show, which was enjoyed by all.

METROPOLITAN SECTION

"Preparing for Public Works in the Post-War Period" was the subject of discussion at the May meeting, which was held at New York University on the 19th. A picnic on the campus, which preceded the technical program, was much enjoyed. Then Frank W. Herring, assistant director of the National Resources Planning Board, analyzed the relation between public works construction and the entire national income. Following a spirited discussion of this talk, Admiral R. E. Bakenhus, chairman of the Section's Committee on Student Chapters, presented the Section's awards of Junior membership in the Society to the outstanding senior in each of the Student Chapters within the Section. The recipients were as follows: Jesse Kapor, College of the City of New York; Wilfred P. Stapelfield, Columbia University; Walter T. Sittner, Cooper Union; Richard J. O'Sullivan, Manhattan College; Walter M. Sura, Newark College of Engineering; Anthony J. Palladino, Rutgers University; and S. Ralph Angell, New York University. The annual election of officers, held at this time, resulted in the selection of Irving V. A. Huie, president; James K. Finch, vice-president; and Charles E. Trout, treasurer. William J. Shea will continue as secretary.

MOHAWK-HUDSON SECTION

An illustrated lecture on laminated timber construction comprised the technical program at the meeting held in Albany on May 21. This was given by Verne L. Ketchum, chief engineer of Timber Structures, Inc., whose talk elicited considerable general discussion.

OREGON SECTION

On May 21 members of the Oregon State College Student Chapter were guests of the Section. The program consisted of the reading of papers in the annual student prize paper contest. Lyle Larsen's paper, entitled "Reynold's Number Applied to Flow in Straight Pipes," was adjudged the most worthy of receiving the award, though it was finally decided that two others also deserved prizes. The latter recipients are William L. Wales for his paper on "Calibration and Use of a Pitot Tube," and Robert C. Ewing for a paper on the solution of simultaneous linear equations.

PROVIDENCE SECTION

During the annual business meeting of the Section, which was held on June 3, new officers were elected for the coming year. These are Daniel O. Cargill, president; Leighton T. Bohl, vice-president; and Frederick H. Paulson, secretary-treasurer. The after-dinner speakers were C. M. Spofford, Vice-President of the Society, and C. B. Breed, Director. The former discussed his recent trip to Dallas and the West Coast, while Mr. Breed's subject was "Air Transport After the War, and Its Effect on Other Forms of Travel."

SACRAMENTO SECTION

Talent from within the Section held the floor at the first three May meetings. On the 4th, John G. Meyer discussed priorities from the viewpoint of the California Division of Highways. On the 11th, the Speakers' Club demonstrated its progress by presenting a group as officers and committees of the two thousandth meeting of the Section, to be held in 1962. The group was portrayed as looking backward on the postwar period. C. D. Conway gave an illustrated lecture on flood control problems along the Sacramento River in Tehama County on May 18.

SAN DIEGO SECTION

At the regular meeting, held on May 27, the guest speaker was William Quon, an electrical engineer employed in aircraft production. Mr. Quon spoke on "The Industrialization of China," giving a brief résumé of the history of that country, its relationship with other nations, present situation, and outlook for the postwar period.

SAN FRANCISCO SECTION

On June 15 Vincent A. Arena, of Stanford University, was a dinner guest of the Section. Mr. Arena is this year's recipient of the Section's award of Junior membership in the Society. A symposium on "The Permanente Magnesium Plant" comprised the technical program, the participants being Peter C. Allen, of the public relations department of the Permanente Metals Corporation, and W. O. Wetmore, metallurgist. A special dinner meeting was held on May 14 to honor members of the National Advisory Committee on Long-Span Suspension Bridges. Activities of the committee, which is making field investigations of the effects of aero-dynamic forces on the stability of long-span suspension bridges and is witnessing tests being conducted on models and prototypes throughout the country, were described by E. F. Kelley, O. H. Ammann, R. B. McMinn, and L. H. Nishkian.

At the May 27 meeting of the Junior Forum Robert M. Kennedy spoke on "Experiences of an Engineer in Panama." The topic for general discussion was "Should We Attempt to Defeat Germany or Japan First?"

TENNESSEE VALLEY SECTION

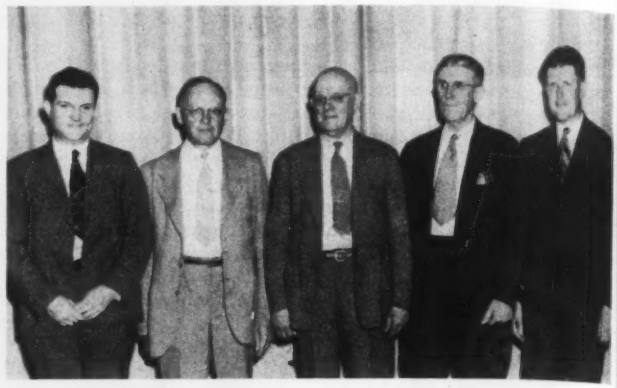
The Knoxville Sub-Section of the Tennessee Valley Section reports meetings in both May and June. At the first of these the members heard R. Brooks Taylor, of the Commerce Department of the TVA, speak on "Crystal Gazing at Technology," in which he developed the theme of the future possibilities of plastics, kaolins, and other materials. The June meeting was addressed by William C. Fitts, general counsel for the TVA.

The Asheville Sub-Section met with the Engineers' Club of Western North Carolina in both May and June. On the first of these occasions Roy Cawhern demonstrated the "Precipitron, an Electronic Air Cleaner," and a motion picture—"Let's Show Them"—was presented. The program was sponsored by the Westinghouse Electric and Manufacturing Company. The June meeting took the form of a banquet and dance.

On May 30 members of the Chattanooga Sub-Section enjoyed a picnic and boat trip on Chickamauga Lake, with local groups of the American Society of Mechanical Engineers and the American Institute of Electrical Engineers.

TOLEDO SECTION

Former presidents of the Toledo Section, shown in the accompanying photograph, were honored at the June dinner meeting. Russell W. Abbott, present president of the Section, spoke briefly about each of his predecessors. The speaker of the evening—Homer T. Yaryan, of the Plaskon Division, of the Libbey-Owens-Ford Glass Company—was then introduced and gave a talk on plastic resin adhesives. The war has given great impetus to the development of glues, which are being used in rapidly increasing amounts for all kinds of construction work. It was predicted that in the near future we will see structural steel members glued together.



FORMER PRESIDENTS OF THE TOLEDO SECTION HONORED AT THE JUNE DINNER MEETING

Reading Left to Right: W. H. Hasselbach (1940), H. M. Sharp (1939), A. S. Forster (1932), C. B. Patterson (1937), and P. W. McDonnell (1938)

Preparations for Transactions, 1943

DURING the summer, plans are under way at Society Headquarters for the printing of TRANSACTIONS in the fall. In spite of delays caused by the war, unavoidable staff absences, and other deterrents, these preparations are progressing satisfactorily during the current summer.

Technical papers or symposia are being unified; the discussions with the authors' closures are assembled for each unit; all corrections are incorporated; extensive proofreading is done; the memoirs are likewise assembled; and the printer receives a green light as fast as sections are ready for printing. The various printing and binding operations in a volume of almost 2,000 pages are quite formidable. For this reason, the process begins as soon as the June PROCEEDINGS is out of the way. Indeed, much of the work is done even earlier in the year as completed papers become available. The full machinery is dovetailed to bring about completion in time for mailing out the TRANSACTIONS in paper covers with the October issue of PROCEEDINGS.

In spite of the paper shortage, it has proved feasible to plan a generous TRANSACTIONS for 1943. The prospects are that about 1,750 pages will be printed. What is more important, no essential items that are ready for use will be omitted.

Appointments of Society Representatives

BORIS A. BAKHMETEFF and GLENN L. PARKER, Members Am. Soc. C.E., have been appointed the Society's representatives on the American Association for the Advancement of Science.

RAYMOND A. DAWSON, M. Am. Soc. C.E., has been appointed to represent the Society as Advisory Trustee of the Texas Natural Resources Library, at San Antonio, Tex.

DONALD C. A. DU PLANTIER, M. Am. Soc. C.E., has accepted appointment to the Society's Committee on Student Chapters, representing the Southern region for a term of three years, ending in 1947.

CHARLES E. TROUT, M. Am. Soc. C.E., has been reappointed one of the Society's representatives on the Board of the Engineering Societies Library for the three-year term, October 1943 to October 1946.

H. E. WESSMAN, M. Am. Soc. C.E., has been reappointed to represent the Society on the Research Procedure Committee of the Engineering Foundation for the term, October 1943 to October 1944.

ITEMS OF INTEREST

About Engineers and Engineering

Construction of an Ancient Dome

By J. Charles Rathbun, M. Am. Soc. C.E.

Professor of Civil Engineering, College of the City of New York, New York, N.Y.

In January 532 A.D., an unruly mob in Constantinople burned the city and endangered the life of the Emporer Justinian. He decided to flee, but his wife, Theodora, prefered to be a wife, widow or corpse, as the fates might decide, instead of a grass widow or exile. She furnished the necessary courage and he took the credit for their bravery. They secured the services of an engineer and started at once to rebuild. Anthemios of Thales designed and built the present structure of Sancta Sophia. He was the ablest man of his day and the most experienced in the art of dome construction. The design of this church was but an extension of this experience to a vastly larger structure than the world had ever known. He used his former successes as models for the new enterprise.

The conception of Sancta Sophia's design was simple. Anthemios would design a dome of the greatest possible span, said dome to be a portion of a sphere of the longest possible radius. The whole was to be mounted, at a great height, on the least possible support.

The usual engineering practice was followed, the Emperor taking all of the credit except for two details. He admitted that an angel gave him the idea of the design, and it was all being done for the glory of the Lord. On the dedication of the church, Justinian exclaimed with due humility, "Solomon I have conquered Thee," while Theodora smiled, Anthemios began looking for another job, the workmen began figuring out their bonuses, and the taxpayers growled. Maybe Justinian was right for he had only one wife to bring him glory and Solomon had plenty of advisers. The fame of the Temple of Solomon was even then legendary, so that no one could dispute him.

The church itself is 250 ft long and 220 ft wide and has a floor area of 11,467 sq yd. The Cathedrals of St. Peter's, Milan, and Seville are the only ones that exceed it in size, and it antedates them all by over a thousand years. It is not the church but its dome that is remarkable. The other great domes of the world are not flat and so present a simpler problem, and there are very few of greater span than Sancta Sophia's.

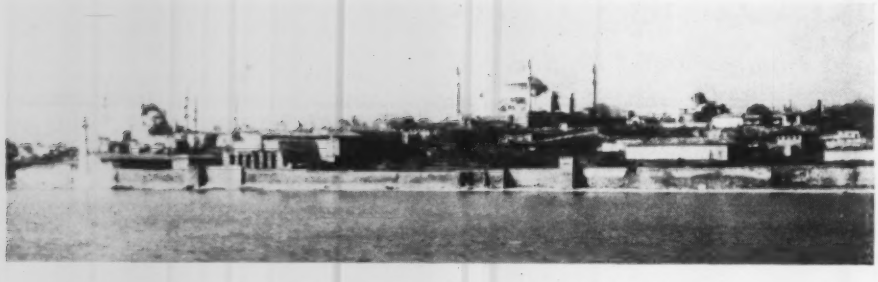
This dome has a span of 108 ft and a rise of only 46 ft above the springing line. The highest point is 180 ft above the floor. That Anthemios was dangerously close to his limit of span and radius of sphere is shown by the fact that the dome fell once a few years after it was built, and it probably fell once during construction. It is supported by eight piers 70 ft high, with arches between them. These piers are surmounted by a circular section containing many windows. This section forms the support for the dome.

To form the dome, tiles varying from 27 by 27 in. to 24 by 24 in., and 2 in. thick, were laid in mortar of nearly the same thickness. These tiles were placed somewhat flatter than radial "to relieve the horizontal thrust at the haunches." Just how this worked out was not recorded by the stress analyst of the day. In construction, the tiles were laid in circular courses starting at the haunch, each course being completed before the next was begun. As the work cantilevered out, the radial pressure was relied on to secure stability. Except for the final closure, the only falsework used was a movable pole pivoted at the center of the spherical intrados. This was more of a guide than a support.

And as to courage, how about that of the tile layers? Trouble was encountered during construction and the dome was finished with tile of but one fifth the normal weight. Light tile were also used in the repair of the damage due to an earthquake in 558 A.D. The horizontal thrust of this very flat dome is taken care of on the north and south faces by semi-domes resting on curved walls and pressing against the building. The same purpose is accomplished by buttresses on the east and west faces.

Much that is hard to believe can be learned about all very old structures. Sancta Sophia is no exception. It is recounted that, after the damage of 558 A.D. was repaired, the church was flooded to a depth of five cubits of water in order to avoid jarring the church when the falsework was dropped. That's a lot of water. One can learn, furthermore, that the mortar of the dome is mixed with sand from Mecca. Those who bring up in objection the mere details that Mecca did not have much fame (but did have plenty of sand) in 532 A.D., or even 558 A.D., and that the builders, being Christians, would not need recourse to Mecca, still would have to face the problem of why the dome stood so well during the centuries that the building was a mosque. Then there are

Sancta Sophia, 1400-Year-Old Structure, Now a Government Museum



THE GOLDEN HORN, FROM ACROSS THE BOSPHORUS

Sancta Sophia in Center and, at Left, the Blue Mosque of Achmet I, Built 1609–1614 as a Rival to It (See Item, January 1941 Issue, Page 69)

the three doors made of wood from Noah's Ark. Any one rubbing these doors is immune from shipwreck. The writer rubbed them well and has not been shipwrecked since. Furthermore, there are thousands of people still living who could bear the same testimony. During the construction of the church it was often found that angels had done a day's work between the time the workmen laid off at night and came back in the morning. This may account for the fact that the church was finished in less than six years. Another explanation is that Anthemios was an able organizer and Justinian knew how to levy taxes. Many more stories are told, but the main historical facts are a matter of authentic record.

The history of the edifice is as interesting as its engineering features. It was started on Feb 23, 532 A.D., and dedicated December 26, 537. It was pillaged by the Crusaders in 1204, when they stole the horses now on St. Mark's in Venice and one of the skulls of John the Baptist. It became a Mohammedan Mosque in May of 1453. It is now a government museum.

Although everything that could be moved was stolen by the Mohammedans or the Crusaders (the latter got there first) the structure is practically the same as it was in ancient times. The mosaics were painted over by the Mohammedans as this was easier than destroying them. They are gradually being brought back to view.

Sancta Sophia stands as a monument to the ability of Anthemios, who anticipated and provided for the various stresses induced in this complicated structure. This is astonishing when one considers that Galileo a thousand years later struggled unsuccessfully with the analysis of the simple beam.

N. G. Neare's Column

Conducted by R. ROBINSON ROWE, M. AM. SOC. C.E.

Enroute to Los Angeles Convention July 25, 1943

"Mr. G. C. Berry, "Secretary, Engineers' Club "Esseyeville, U.S.A.

"Dear Mr. Secretary: As I will not return from the Annual Convention until after our August meeting, I wish you would read this letter to the Club, assuring dissenters of their innings in September.

"The June problem required the spread of the personal fortunes of two billion people so that, if each in turn distributed half his instant holding equally among the others, the final holdings would be alike. Several correspondents deduced general equations of the following or equivalent form, in which p is the population and n the order of an individual's distribution. If all transactions are integral, each final holding must be $(p-1)^p$ or some multiple thereof, and the initial fortune of the nth man must have been:

$$F_n = (p-2)p^{-n}[p(p-1)^{n-1} + (p-2)^n]$$

The spread, or ratio of Zyzyx's fortune to Aab's, was:

$$\frac{F_p}{F_1} = \frac{p(p-1)^{p-1} + (p-2)^p}{p(p-2)^{p-1} + (p-2)^p} = \frac{\left(\frac{p-1}{p-2}\right)^{p-1} + 1 - \frac{2}{p}}{2 - \frac{2}{p}}$$

$$= \frac{e+1}{2} = 1.85914 \qquad p \to \infty$$

"For the given population, the spread is sensibly the same; of the total wealth of 200 billion dollars, Aab had about $73.58 and Zyzyx about $136.79. I say 'about' because the transactions are not integral in cents; for the second part of the problem, Tom Dowtz was asked to adapt the postwar yen to the purpose and to compute its exchange value, y, in cents. Obviously,

$$y = \frac{2 \cdot 100 \cdot 10^{11}}{p(p-1)^p} = \frac{10{,}000}{1{,}999{,}999{,}999^{2{,}000{,}000{,}000}}$$

which is somewhere between Kasner's googolth and his googolplexth. In exchange, the cent will buy 7 828 . . . 600 000 000 000 000.0001 postwar yen, in which number the dots replace 18 602-059 968 digits which Tom didn't compute. Please don't ask 'Why?'

"As we left Esseyeville yesterday, Ken Bridgewater bought a Cowtown Transcript that had suffered a pressroom accident. One of the major league standings looked like this:

	W	L	Per
Antville			0.750
Beeburgh			0.625
Cowtown			0.600
Dogland			0.500
Elk Rapids			0.450
Fleaborough			0.444
Gnatopolis			0.385
Hippodelphia			0.200

"My Antville Tidings was intact, and I remarked to Ken the coincidence that the percentages (or permillages) were exact. Because of rain, some teams were behind schedule, but none more than one game in seven. Antville had played the most and was 17 games ahead of Beeburgh. Hippodelphia had played the least and trailed Gnatopolis by 20. How many games had been played in the league?

"See you in September.

"NOAH G. NEARE"

[The able correspondents were James R. Bole and Robert W. Woodbury, both from California, who rediscovered the general solution of Richard Jenney acknowledged in June.]

NEWS OF ENGINEERS

Personal Items About Society Members

ROBERT L. SUMWALT, since 1930 professor of civil engineering at the University of South Carolina, has been elected dean of the engineering school there. He replaces WALTER E. ROWE, who recently retired after twenty-one years in the capacity of dean.

RALPH A. SMITH, formerly chief engineer of the Holyoke (Mass.) Water Power Company, has been appointed production superintendent of the Rockland County Light and Power Company, with headquarters at Nyack, N.Y. Mr. Smith will have charge of operation and maintenance of 16 generating plants and 500 miles of transmission line.

JAMES B. HAYS is in New York City engaged in preparing plans for post-war development by means of irrigation and hydroelectric power in the Near East. Until lately Mr. Hays was project manager on the South Holston and Watauga projects of the TVA at Bristol, Tenn.

ROBERT M. KNOX has been made president and treasurer of A. Guthrie and Company, Inc. He formerly held the position of vice-president.

PHILIP C. RUTLEDGE recently accepted a professorship in civil engineering at the new Technological Institute of Northwestern University at Evanston, Ill. For the past six years Professor Rutledge has been successively associate professor and professor of soil mechanics at Purdue University.

LEE G. WARREN announces his resignation as project manager for TVA after eight years in the service of the Authority. Following a vacation, Mr. Warren plans to join a contracting organization.

ROGER C. BROWN and FRANK RAGAINI have become associated with CLARENCE M. BLAIR, of New Haven, Conn., in the consulting firm of Clarence M. Blair, Inc.

MARVIN O. KRUSE is now a major in the Corps of Engineers, U.S. Army. Major Kruse has been serving in the armed forces since early in 1941 and has had a long and active record in such front-line areas as New Guinea. In the June issue it was incorrectly stated that Major Kruse had just joined the Corps of Engineers.

JOSEPH A. JORDAN, of New York City, has accepted a position as general superintendent of the McAvoy Shipbuilding Company, with headquarters at Savannah, Ga. He will be engaged on the construction of concrete boats.

E. WARREN BOWDEN, previously with Parsons, Klapp, Brinckerhoff and Douglas, of New York City, has accepted a new connection as assistant to the president of Walter Kidde Constructors, Inc., of New York.

L. A. PETERMAN is back with the Texas State Highway Department after two and a half years on emergency war work in Texas and Louisiana. Stationed at Navasota, Tex., he is working on postwar plans for the Department.

CHARLES E. NICHOLS has severed his connection with the TVA in Washington, D.C., in order to become Washington representative of the National Warehousemen's Association. Mr. Nichols will be succeeded in his role as head of the Priorities Office of the Authority in Washington by J. M. CHESTON.

R. O. DAVIS has been promoted from the rank of lieutenant in the U.S. Army to that of captain. He is in the Aeronautical Chart Division of the Air Forces, with headquarters in St. Louis, Mo.

C. B. MCGAVOCK, JR., formerly associate geologist for the TVA at Knoxville, Tenn., has been sent to Brazil by the Board of Economic Warfare. He is located in Rio de Janeiro.

LORAN D. GAYTON, assistant city engineer of Chicago, Ill., received the honorary degree of C.E. from the University of Illinois during the past commencement season.

FRED C. TAYLOR was recently promoted from the rank of major in the U.S. Army to that of lieutenant colonel and assigned to Army Service Forces, New York City, where he is Chief, Transportation of Persons, Transportation Corps of the U.S. Army. Colonel Taylor was previously with the Training Circular of the Army in Washington, D.C.

FREDERIC BASS, formerly head of the department of civil engineering at the University of Minnesota, has been appointed executive director of the American Public Works Association.

ROBERT R. HARRIS, captain, Corps of Engineers, U.S. Army, has been assigned to the Office of the Coordinator of Inter-American Affairs at Managua, Nicaragua. He was previously assistant sanitary engineer for the Kentucky State Department of Health.

C. C. CHAMBERS has resigned as chief engineer for the Muskingum Watershed Conservancy District at New Philadelphia, Ohio, but will be retained as consulting engineer for the District.

CHARLES E. CARL, until lately health engineer for the Jackson County (Missouri) Health Department, has been commissioned a first lieutenant in the Sanitary Corps of the U.S. Army. He is stationed at the Jackson (Miss.) Army Air Base.

WILLIAM STATES LEE, JR., was recently promoted from the rank of lieutenant commander in the U.S. Navy to that of commander. Commander Lee heads the 15th Naval Construction Battalion in the South Pacific.

THOMAS B. SCHULTZ is now a second lieutenant in the U.S. Army, with headquarters at Camp Barkeley, Texas. Before being commissioned he was sanitary engineer for the Pennington County (South Dakota) Health Department.

WALDO G. BOWMAN, editor of *Engineering News Record*, left at the end of May for a three-month journey to the British Isles, Africa, and the Near East. Traveling as a war correspondent, Mr. Bowman is also engaged on a special mission concerned with Army and Navy civil engineering activities.

SAMUEL B. MORRIS, dean of the school of engineering at Stanford University, was recently elected president of the American Water Works Association.

ARTHUR H. BLANCHARD, after completing nine months' work on the design and fabrication of a war machine now on every battle front, is serving as a metallurgical engineer for Breeze Corporations, Inc., Newark, N.J.

DECEASED

FRANCIS OGDEN BLACKWELL (M. '04) president of Viele, Blackwell and Buck, of New York, N.Y., died suddenly at his home in Englewood, N.J., on April 11, 1943. Mr. Blackwell, who was 76, had been with the firm of Viele, Blackwell and Buck for over thirty years. Earlier in his career he was in charge of the construction of electric street railways for various New England cities, and at one time acted as consultant to the Mexican Light and Power Company and the Electrical Development Company, of Ontario.

SYLVESTER QUALE CANNON (M. '17) consulting engineer of Salt Lake City, Utah, died there on May 29, 1943, at the age of 67. Mr. Cannon had been state engineer in charge of the Weber River irrigation survey, and from 1913 to 1925 was city engineer of Salt Lake City. At various times he maintained an engineering practice in Utah and was widely known for his activities in city paving, water works, and sewage disposal. Long prominent in Mormon Church circles, Mr. Cannon was for thirteen years presiding bishop of the Church of Latter Day Saints.

RICHARD ALESTER COLEMAN (Jun. '40) for the past year and a half in the technical division of the Newport News Shipbuilding and Dry Dock Company, Newport News, Va., died in a hospital there on December 16, 1942. Mr. Coleman, who was 22, received the degree of B.S. in civil engineering from Worcester Polytechnic Institute in 1940.

FREDERICK MUSSEN GAHAGAN (Assoc. M. '31) president of the Gahagan Construction Corporation, of Brooklyn, N.Y., died in New York City on July 7, 1943, at the age of 44. Several months ago Mr. Gahagan returned from Panama where, for the past two years, he had been overseeing projects for his company. In 1923 he joined the construction firm headed by his father, the late Walter H. Gahagan, as assistant engineer. Since his election to the presidency of the company in 1933, Mr. Gahagan had supervised the direction of contract operations for many federal, state, and municipal projects, including the Ward's Island sewage disposal plant.

JULES DUCI GEDO (M. '35) structural designer for Alexander D. Crosett of New York, N.Y., died on February 24, 1942, though word of his death has just reached the Society. He was 49. Born and educated in Hungary, Mr. Gedo spent his early professional career there and in Czechoslovakia and Germany. Coming to the United States in 1924, he became bridge detailer for the American Bridge Company at Gary, Ind., and later was with the New York consulting firm of Purdy and Henderson. Except for a period with the New York City Department of Parks, he had been with Alexander D. Crosett since 1928.

IRVIN GITTLESON (Assoc. M. '35) of New York, N.Y., died on June 11, 1943, at the age of 40. From 1922 to 1926 Mr. Gittleson was draftsman for the Virginia Bridge and Iron Company at Roanoke, Va., and from the latter year to 1933 he was employed in a similar capacity by Waddell and Hardesty, of New York City. Later he was with other New York firms, including Ash-Howard-Needles and Tammen.

WILLIAM HERZBERG (M. '42) chief construction engineer for the Alabama State Highway Department at Montgomery, Ala., died in Birmingham on June 4, 1943. Mr. Herzeberg, who was 53, had been county engineer of Knoxubee County, Mississippi, and city engineer of Erwin, Tenn. Later (1919 to 1921) he was with the Georgia State Highway Department, and since 1921 had been with the Alabama State Highway Department—from 1939 on as state construction engineer.

BEN STUART KIMMELSMAN (Jun. '42) died suddenly on April 14, 1943, at Fort Belvoir, Va., where he was in the Corps of Engineers, U.S. Army. Mr. Kimmelsman graduated from the O.C.S. School at Fort Belvoir as a second lieutenant and was to have received his bars on the day of his death. Before joining the Army Mr. Kimmelsman was with the Henry J. Kaiser Company at Oakland and, later, at Fontana, Calif. He was an alumnus of the University of California, class of 1942.

ALEXANDER ANDREW LAIRD (Assoc. M. '20) colonel, Corps of Engineers, U.S. Army, Phoenix, Ariz., died in a hospital in Denver, Colo., on May 18, 1943. For the past three years Colonel Laird had

been liaison officer for the Under-Secretary of War to the Arizona Selective Service headquarters at Phoenix. An Army veteran of thirty years' service—most of it with the Corps of Engineers—he was with the 315th Engineers, 90th Division, during the first World War.

WILLIAM JOHNSTON MITCHELL (Assoc. M.'31) of Fairhaven, Mass., died at St. Petersburg, Fla., on May 19, 1943, at the age of 52. Mr. Mitchell had been with the American Bridge Company, and the Berlin (Conn.) Construction Company, and for ten years was designer in the engineering department of the Bethlehem (Pa.) Mines Corporation. During the first World War Mr. Mitchell served with the 29th Engineers, U.S. Army, in France.

ROGER WILLIAMS PARKHURST (Assoc. M. '21) who was with the Mechanical Equipment Division of the Allied Works Council, with headquarters in Melbourne, Australia, was killed in May, 1943 when the airplane in which he was traveling was shot down in New Guinea. Mr. Parkhurst, who was 52, spent his early career in railroad work in this country. Beginning in 1920, he was for a number of years engineering representative for the Barber Asphalt Company, of Philadelphia, in India and Australasia, successively. During the first World War Mr. Parkhurst served overseas with the 313th Engineers, attaining the rank of captain.

GUY BENNETT WAITE (M. '07) a pioneer in concrete building construction, died at his home at Belle Harbor (Queens), N.Y., on June 26, 1943. His age was 81. Early in his career Mr. Waite taught at his alma mater, Renesselaer Polytechnic Institute. Until 1920 he was identified with the erection of many of New York's large buildings. He developed and patented many ideas in building construction and other fields.

ALBERT WILLARD WALKER (Assoc. M. '10) superintendent for the U.S. Bureau of Reclamation at Fairfield, Mont., died at Great Falls, Mont., on June 15, 1943. Mr. Walker, who was 61, had spent most of his career with the Bureau of Reclamation, having become connected with its predecessor, the U.S. Reclamation Service, in 1905. He served as construction engineer on numerous projects for the Bureau, including the Belle Fourche Project.

JAMES MELLICK WILCOX (Assoc. M. '23) of Weehawken, N.J., died on January 24, 1943, at the age of 47. Mr. Wilcox had been in Chile with the Chile Exploration Company and in Egypt with the Anglo Egyptian Company. He had also been designing engineer for the Teaneck (N.J.) Department of Public Works. During the first World War Mr. Wilcox served as a lieutenant (jg) on various ships.

Changes in Membership Grades

Additions, Transfers, Reinstatements, and Resignations

From June 10 to July 9, 1943, Inclusive

ADDITIONS TO MEMBERSHIP

ANDERSON, WILLIAM JOSEPH (Jun. '43), Junior Test Engr., Wright Aeronautical Corp., Beckwith Ave. (Res., 174 Franklin St.), Paterson, N.J.

BASSETT, ELLSWORTH WATSON (Assoc. M. '43), Senior Engr. (Civ.), Office, Chf. of Engrs., U.S. Army, New War Dept. Bldg., Washington, D.C. (Res., 2148 North Brandywine St., Arlington, Va.)

BEESON, DONALD RICHARD, JR. (Jun. '43), With Bethlehem Steel Co., Sparrows Point, Md. (Res., 2625 Liberty Parkway, Baltimore 22, Md.)

BISHOP, LOREN CLARK (M. '43), State Engr., 2112 Maxwell, Cheyenne, Wyo.

BISSELL, ARTHUR DOUGLAS, 2D (Jun. '43), 80 South Cayuga Rd., Williamsville, N.Y.

BOWSER, CLARENCE DEWEY (M. '43), Designing Engr., Bureau of Bridges and Grade Separations, State Dept. of Highways, 66 South Front St., Columbus 15 (Res., 1563 Genessee Ave., Columbus 3), Ohio.

BRADDOCK, EDWARD (Assoc. M. '43), Supervisor, Roads and Grounds Dept., Twin Cities Ordnance Plant, New Brighton (Res., 4201 Drew Ave., South, Minneapolis), Minn.

BRANAHL, ERWIN FRED (June. '43), 4212a Athlone Ave., St. Louis, Mo.

BRANDT, CHARLES EDWARD (Jun. '43), 2d Lt., Field Artillery, U.S. Army, 3728 Guilford Ave., Indianapolis, Ind.

BYERRUM, SEVEREN ALFRED (Jun. '43), Ensign, CEC-V (S), U.S.N.R., U.S. Naval Air Depot, Hastings, Nebr.

CAMPBELL, WILLIAM JOSEPH (Jun. '43), Procedures Engr., Curtiss-Wright Corp., Robertson (Res., 10123 St., Martha Lane, Overland), Mo.

CARDWELL, THOMAS LLOYD (Assoc. M. '43), Sales Engr., Armco Drainage & Metal Products, Inc., 310 South Michigan, Chicago, Ill.

CHARLES, JACK RANDOLPH (M. '43), Cons. Engr., Basic Magnesium Inc., Las Vegas, Nev.

COBLE, ROY ELVIN (Jun. '43), Ensign, E-N (S), U.S.N.R., 1010 Winchester, Alhambra, Calif.

COLLINSWORTH, EDWARD REX (Jun. '43), Minor Layout Draftsman, Consolidated Vultee Aircraft Corp. (Res., 912 West Terrell, Apt. 3), Fort Worth 4, Tex.

CRUTCHER, JAMES WHITMORE (Jun. '43), Marine Draftsman, Dravo Corp., Ft. of Madison St., Wilmington, Del.

DASHIELL, ROBERT FREELAND (Jun. '43), Erection Engr., Chicago Bridge and Iron Co. (Res., 1922 F St.), Eureka, Calif.

DIKER, VEJDI RIFAT (Jun. '42), Care, Ministry of Public Works, Ankara, Turkey.

DILDINE, ALFRED CARR (Jun. '43), Ensign, CEC-V (S), U.S.N.R., 5801 Adelaide Ave., San Diego, Calif.

DIVEN, JOHN ALLEN (Assoc. M. '43), Junior Bridge Engr., State Roads Comm., 108 East Lexington St. (Res., 507 Murdock Rd.), Baltimore 12, Md.

DOOLITTLE, WILLIAM KARL (Assoc. M. '43), Structural Designer, The Austin Co., 220 West Main St. (Res., 3114 Jefferson St.), Midland, Mich.

ECKSTEIN, GEORGE JOSEPH (Assoc. M. '43), Asst. Civ. Engr., Dept. of Public Works, City of New York, Municipal Bldg. (Res., 735 West 172d St.), New York 32, N.Y.

ELLIOTT, JAMES HENRY (Jun. '42), Ensign, U.S.N., 231 Highland Ave., Greensburg, Pa.

EMERY, ROY WILLIAM (Assoc. M. '43), Design Engr., H. G. Acres & Co., 2135 Culp St. (Res., 2184 Culp St.), Niagra Falls, Canada.

EVANS, PAUL EDWARD (Assoc. M. '43), Lt. (jg), CEC-V (S), U.S.N.R., 3484 Arthur St., San Diego, Calif.

FARMER, THOMAS MORAN (Jun. '43), Junior Structural Engr., Chance Vought Aircraft, Stratford (Res., 433 Golden Hill St., Bridgeport), Conn.

TOTAL MEMBERSHIP AS OF JULY 9, 1943	
Members	5,946
Associate Members	7,253
Corporate Members	13,199
Honorary Members	33
Juniors	5,632
Affiliates	74
Fellows	1
Total	18,939
(Total July 9, 1942	17,877)

FITZPATRICK, JOSEPH WILLIAM (June. '43), Asst. Engr., Fitzpatrick & Weller, Inc., Mill St., Ellicottville, N.Y.

FREIMAN, ALBERT (Jun. '43), Draftsman, Read-York Aircraft, Inc., 5th Ave. and 56th St. (Res., 720 Fifty-ninth Pl.), Chicago, Ill.

FRY, EDWARD, JR. (Jun. '43), 2d Lt., Corps of Engrs., U.S. Army, 82d Engr. Combat Battalion, Camp Swift, Tex.

GERLA, MARVIN (Jun. '43), Junior Stress Engr. Brewster Aeronautical Corp., Johnsville (Res., 219 Hillside Ave., Jenkintown), Pa.

GRAFF, CHARLES RUSSELL (Assoc. M. '43), Supt., Raymond Concrete Pile Co., 816 West 5th St., Los Angeles (Res., 460 Sacramento St., Altadena), Calif.

GUNDER, DWIGHT FRANCIS (Assoc. M. '43), Associate Prof., Civ. Eng., Colorado State College, Fort Collins, Colo.

HAMBY, EDWARD PAUL (Jun. '42), Ensign, U.S.N., Production Div., Norfolk Navy Yard, Portsmouth, Va.

HAMPTON, JAMES EDWARD (Jun. '43), Ensign, U.S.N.R., 10 Sycamore Terrace, Somerville, Mass.

HARDY, CONRAD PALMER (M. '43), Col. Corps of Engrs., U.S. Army, 1313th Engr. Gen. Service Regiment, E.U.T.C., Camp Claiborne, La.

HARNETT, JAMES JOSEPH (Assoc. M. '43), Constr. Supt., Delval Constr. Corp. (Res., 66 Homestead Ave.), Scarsdale, N.Y.

HARPER, DONALD EDWARD (Jun. '43), Stress Analyst, Curtiss-Wright Corp., Lambert Field, St. Louis (Res., 2409 Longfellow Rd., Overland), Mo.

HESTER, HARLAND HARVEY (Assoc. M. '43), Street and Garbage Supt., City of Fort Worth, City Hall (Res., 4014 Mattison Ave.), Fort Worth, Tex.

HILDABRAND, THEODORE JOSEPH (Jun. '43), Dorian Club, Sewickley, Pa.

HODOSH, JACOB (M. '43), Section Engr., New York City Tunnel Authority, 200 Madison Ave. (Res., 304 East 178th St.), New York 57, N.Y.

HOKE, WILBUR CHARLTON (Jun. '43), Eng. Designer, Sanderson & Porter (Res., 1105 Cherry St.), Pine Bluff, Ark.

HOWE, HERBERT EDWARD (Assoc. M. '43), Lt., U.S.N.R., 235 Hillcrest Ave., Fountain City, Tenn.

HUNSICKER, ROBERT LEVINUS (Jun. '43), Stress Analyst, Douglas Aircraft, 702 Olympia Blvd., Santa Monica, Calif.